Toppling - - Failure of Rock Slopes

by Richard E. Goodman

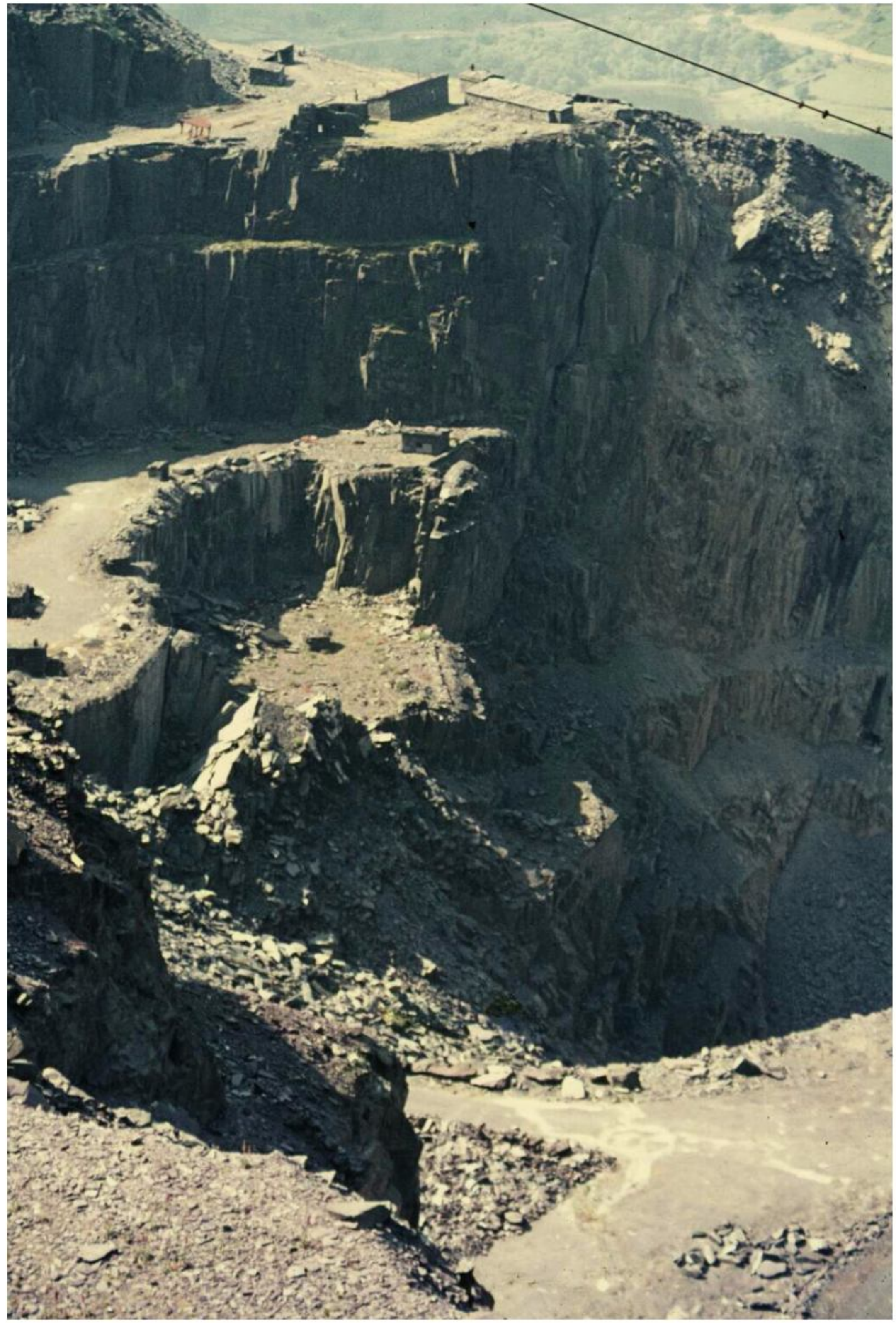


Figure 1, A toppled slope in Dnorwik Slate Quarry, N. Wales, Great Britain

Figure 2 –Aiguille du Midi (Near Mont Blanc)

Mont Blanc –
Aiguille du Midi

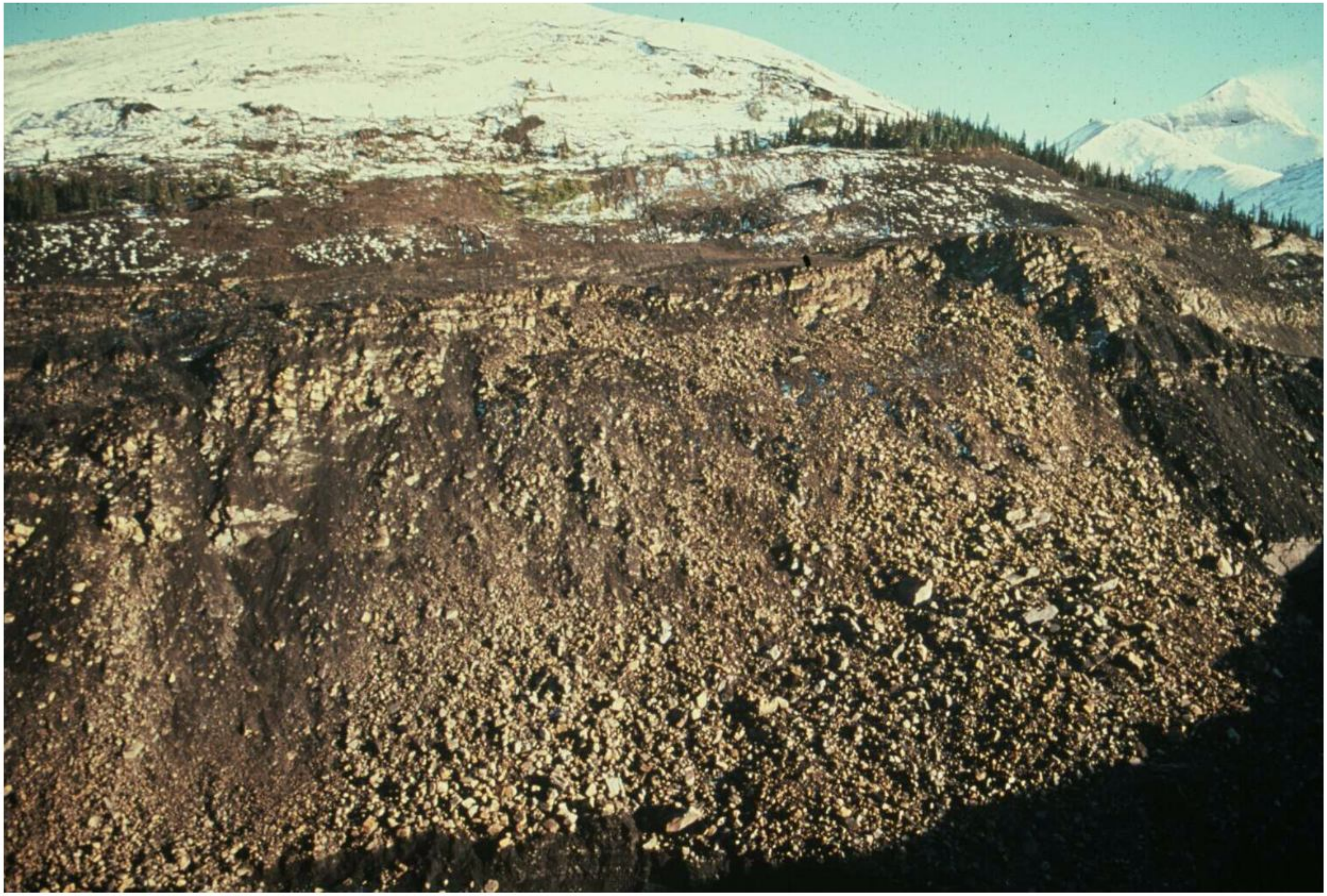
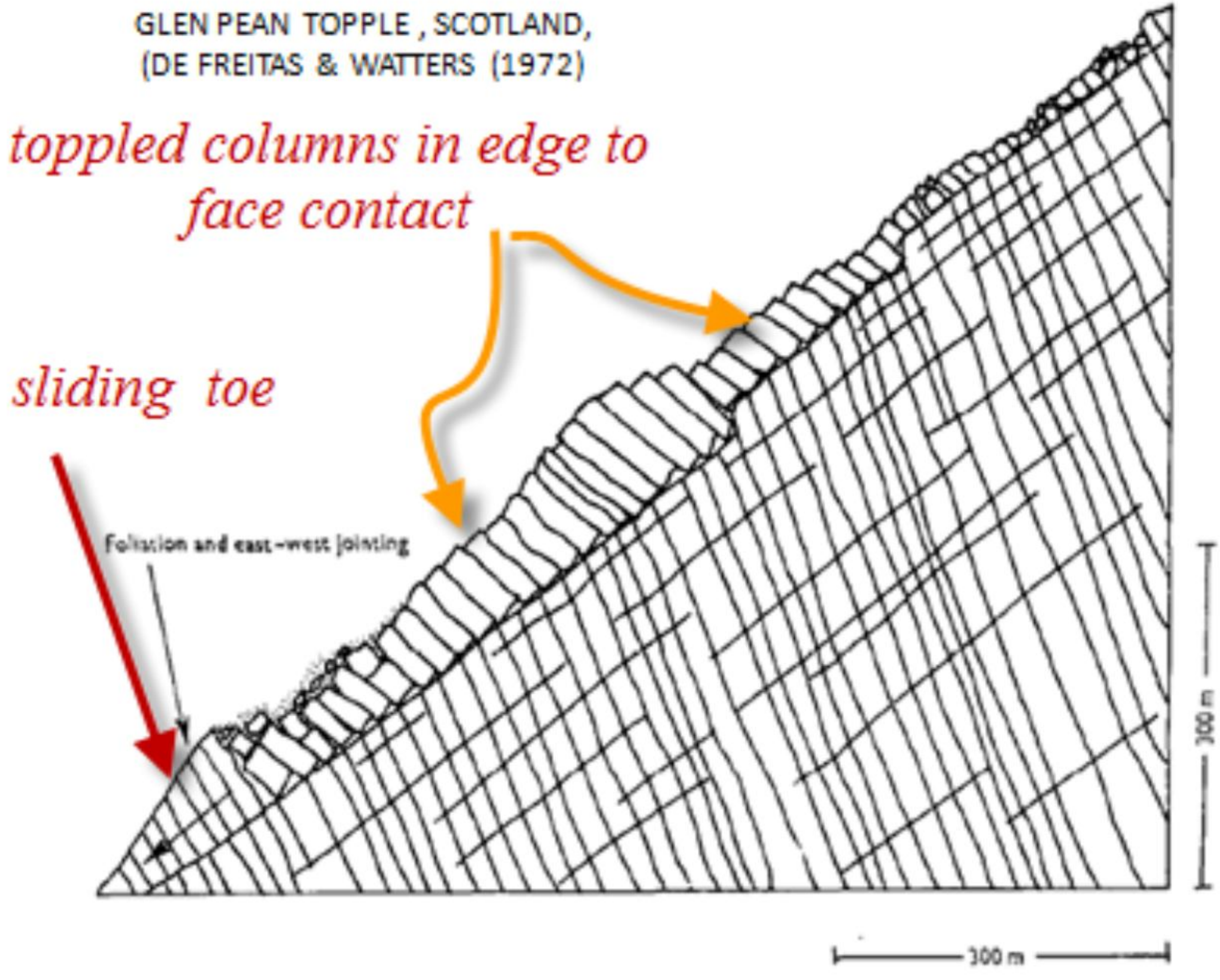
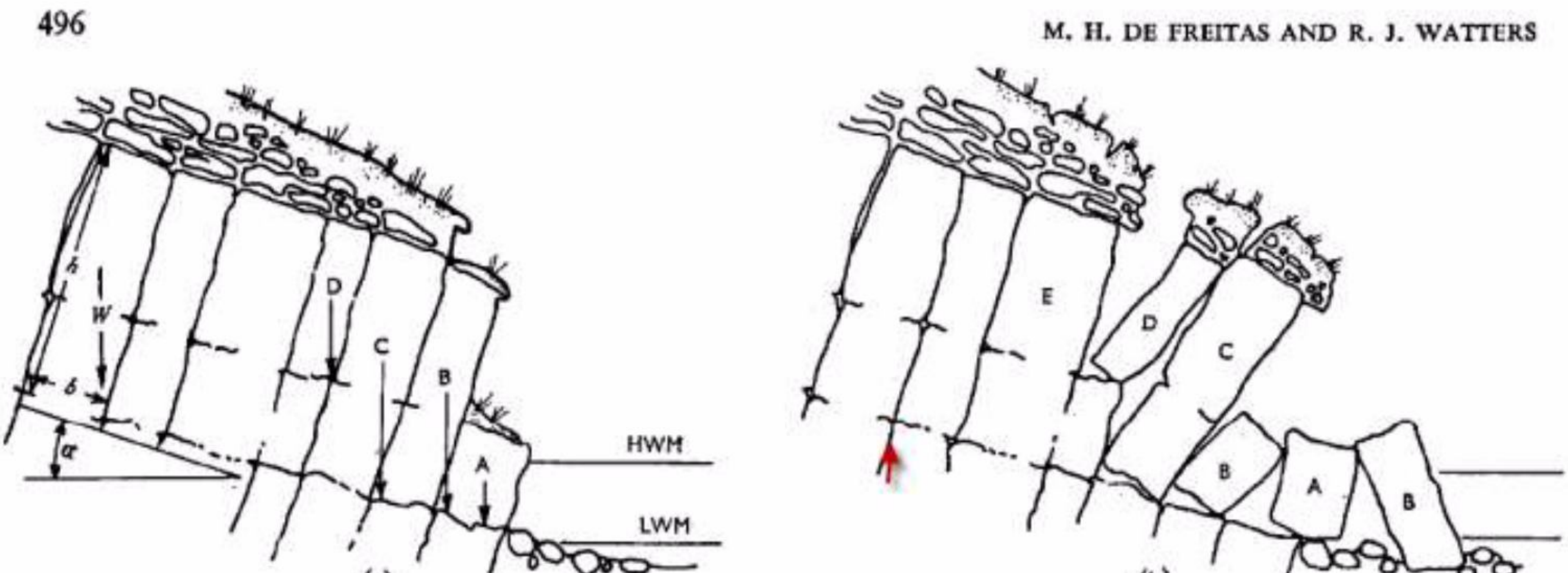


FIGURE 3 – Cardinal River Coal Mine Topple, (Wyllie and Munn, 1978)

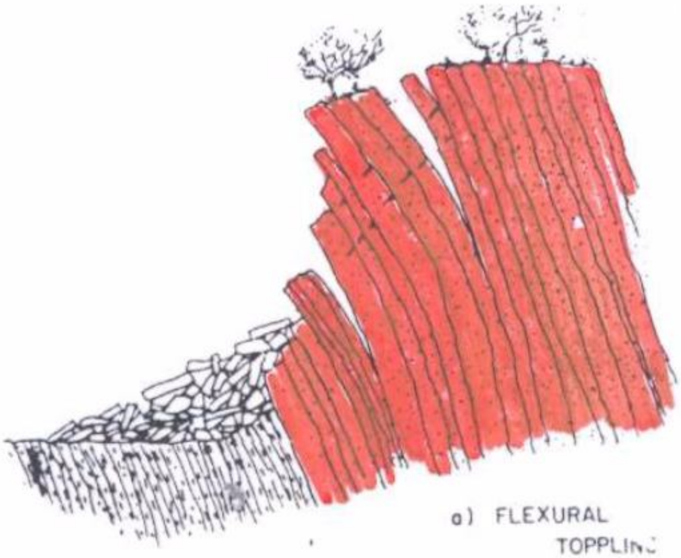
FOLDING distinguished from TOPPLING


GLEN PEAN TOPPLE , SCOTLAND,
(DE FREITAS & WATTERS (1972)
toppled columns in edge to face contact
sliding toe
Foliation and east-west jointing
300 m
300 m

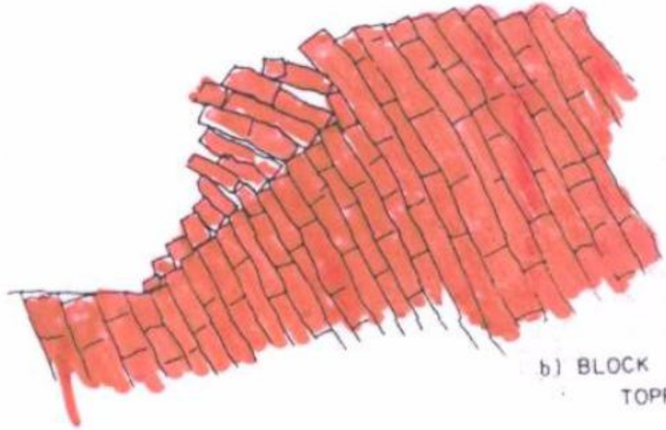
Toppling will occur above a surface inclined at angle α when the ratio b/h $<$tan α. Marine erosion removes Block A, allowing B to topple; Blocks C and D follow but E remains, as it has a wider base.



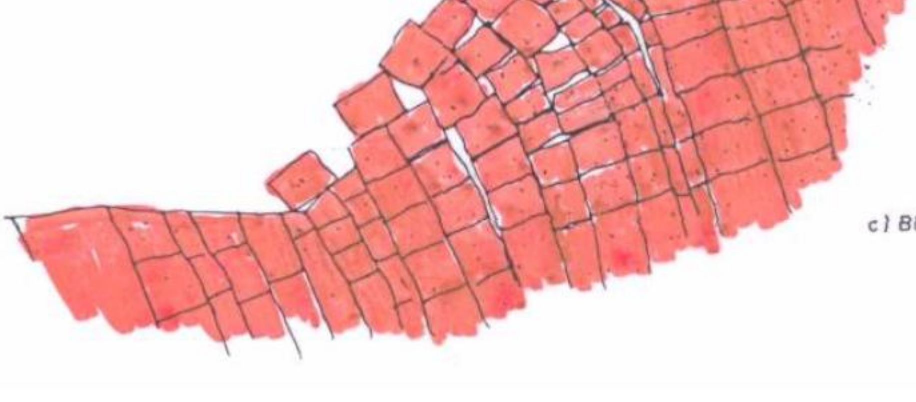
(a)
(b)



a) FLEXURAL TOPPLING

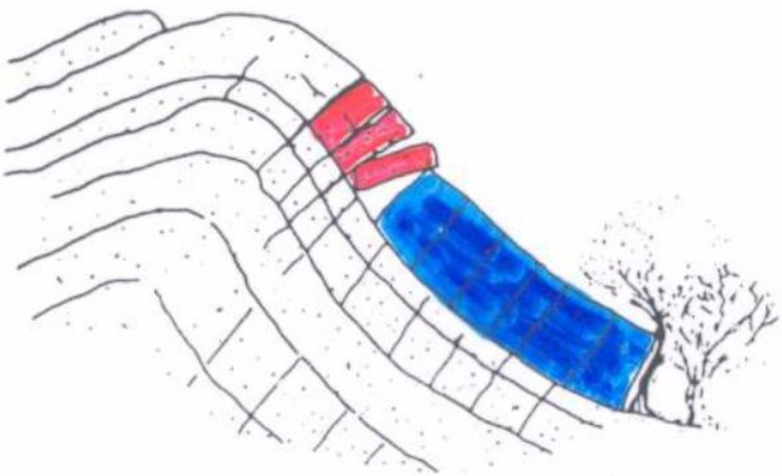


b) BLOCK TOPPLING

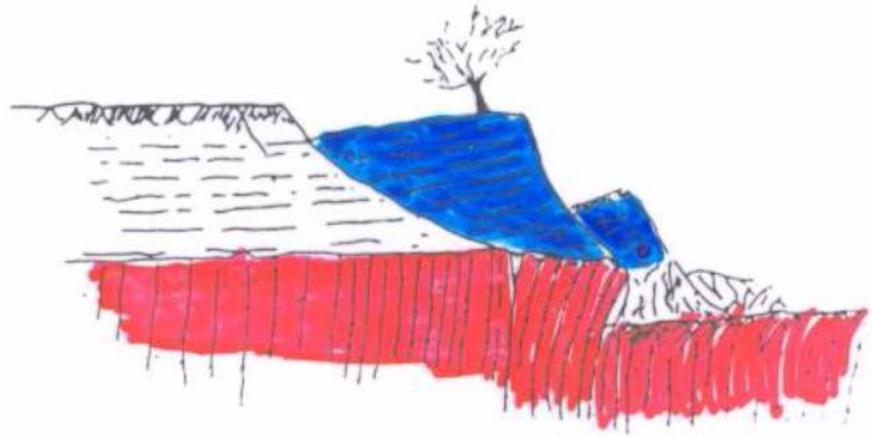


c) BLOCK FLEXURE TOPPLING

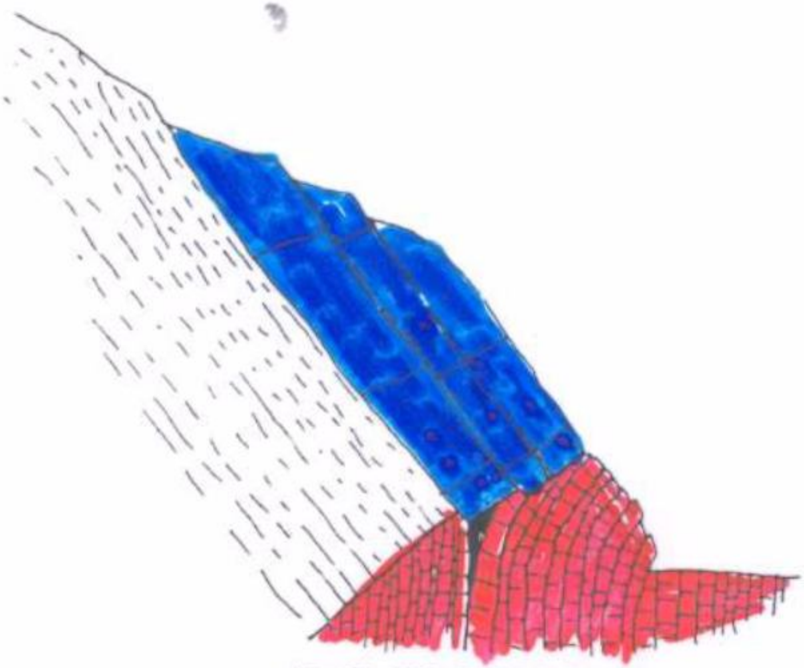
MODES OF "SECONDARY TOPPLING"



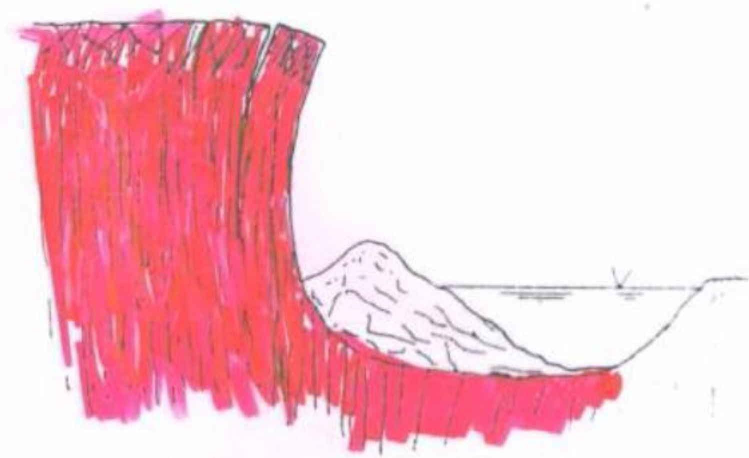
a) SLIDE HEAD TOPPLING



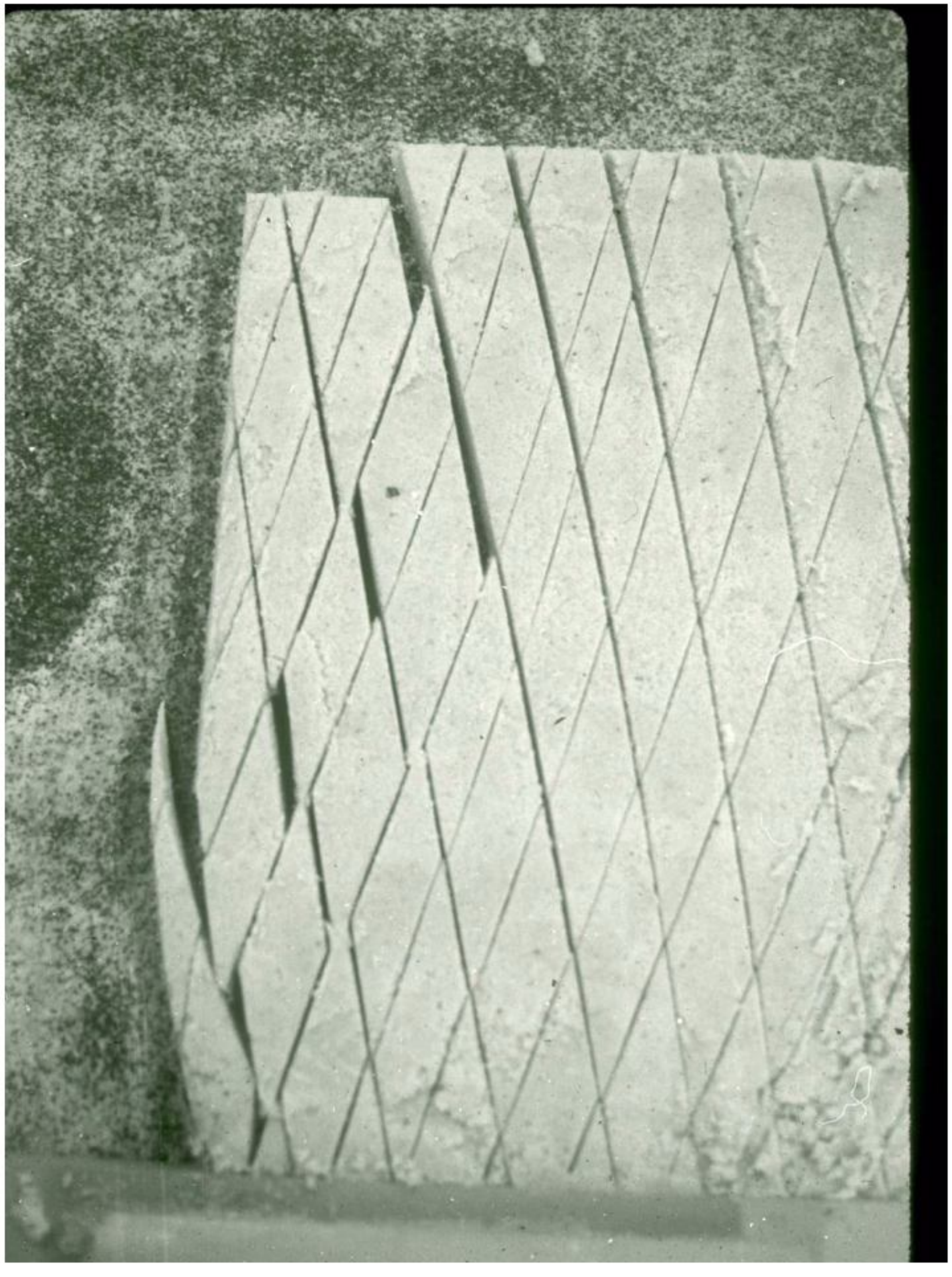
b) SLIDE BASE TOPPLING

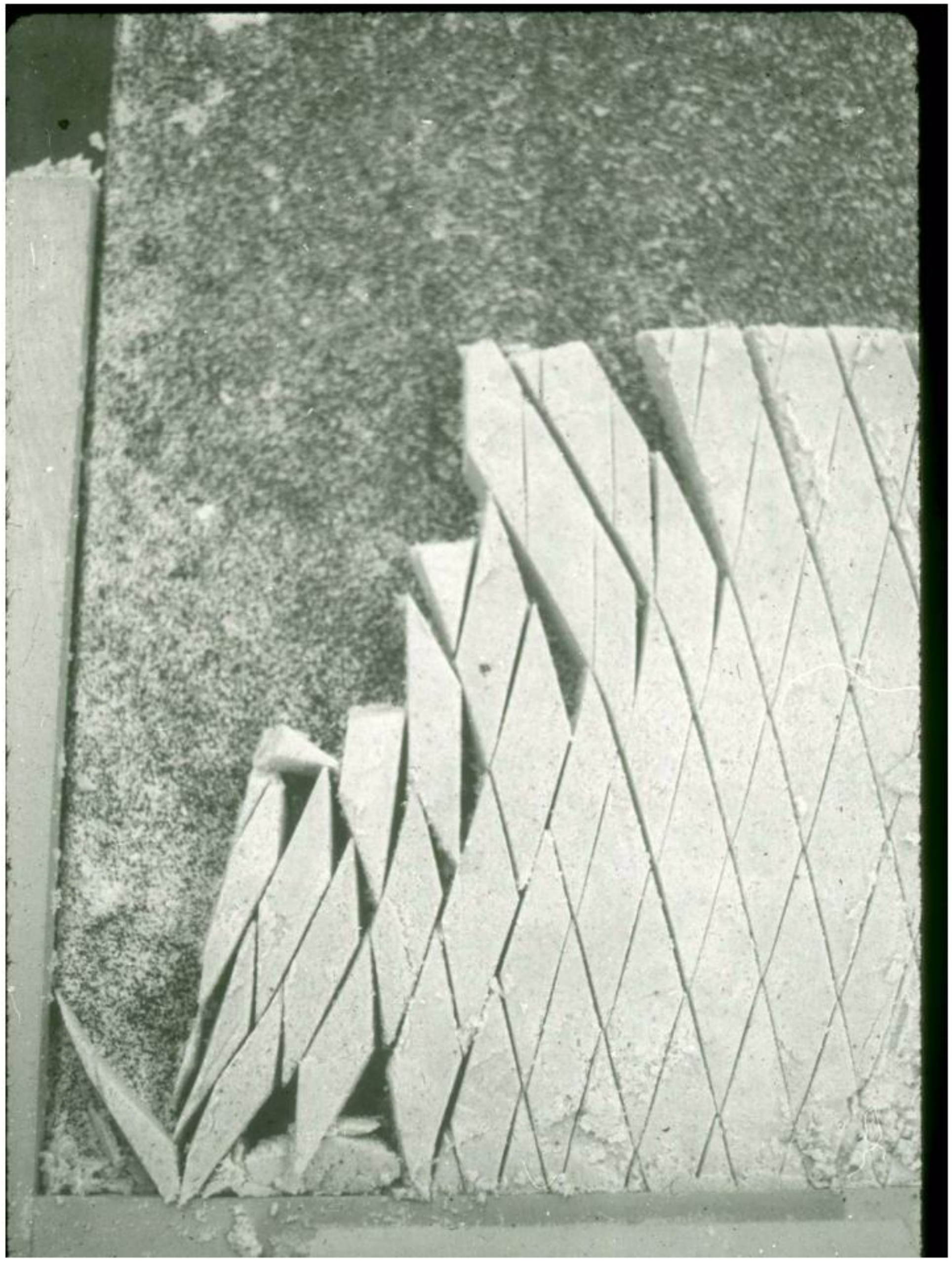


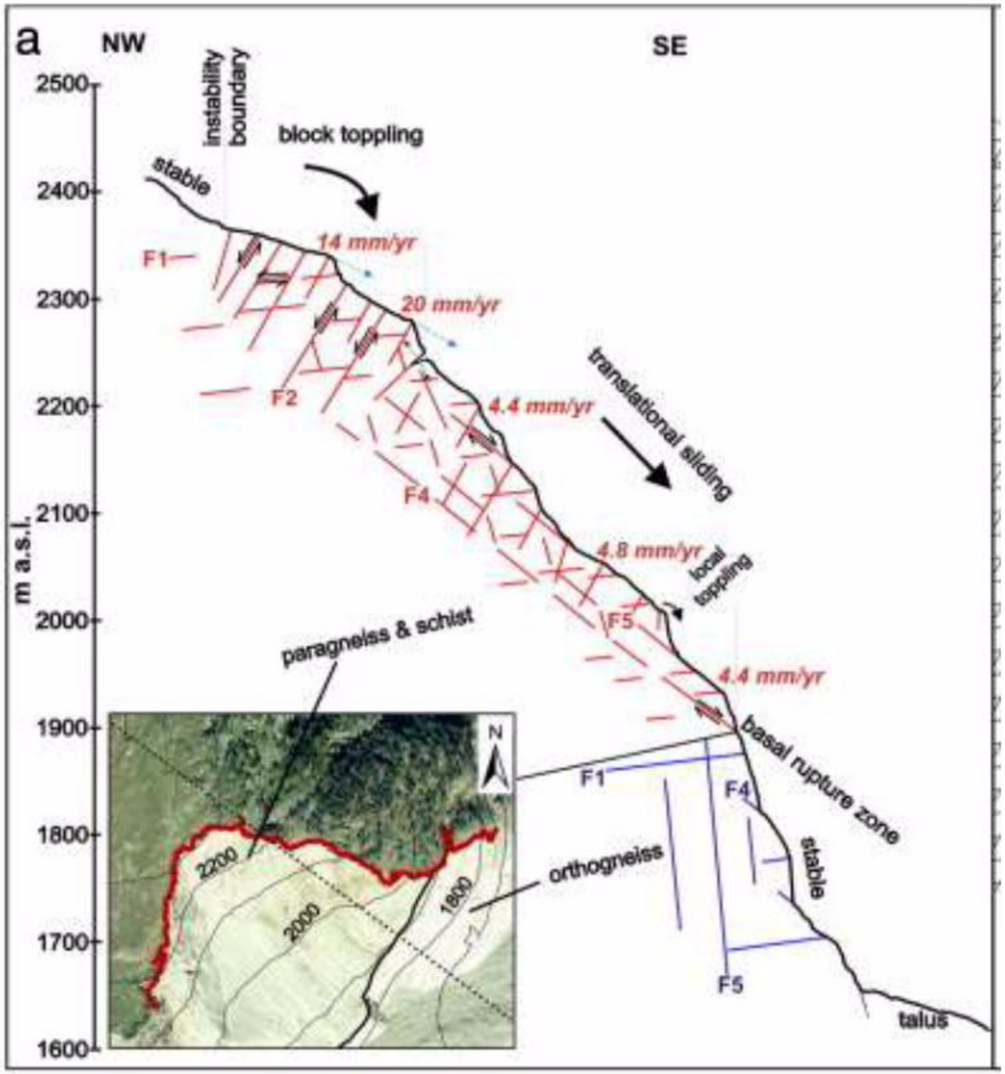
c) SLIDE TOE TOPPLING



d) TENSION CRACK TOPPLING



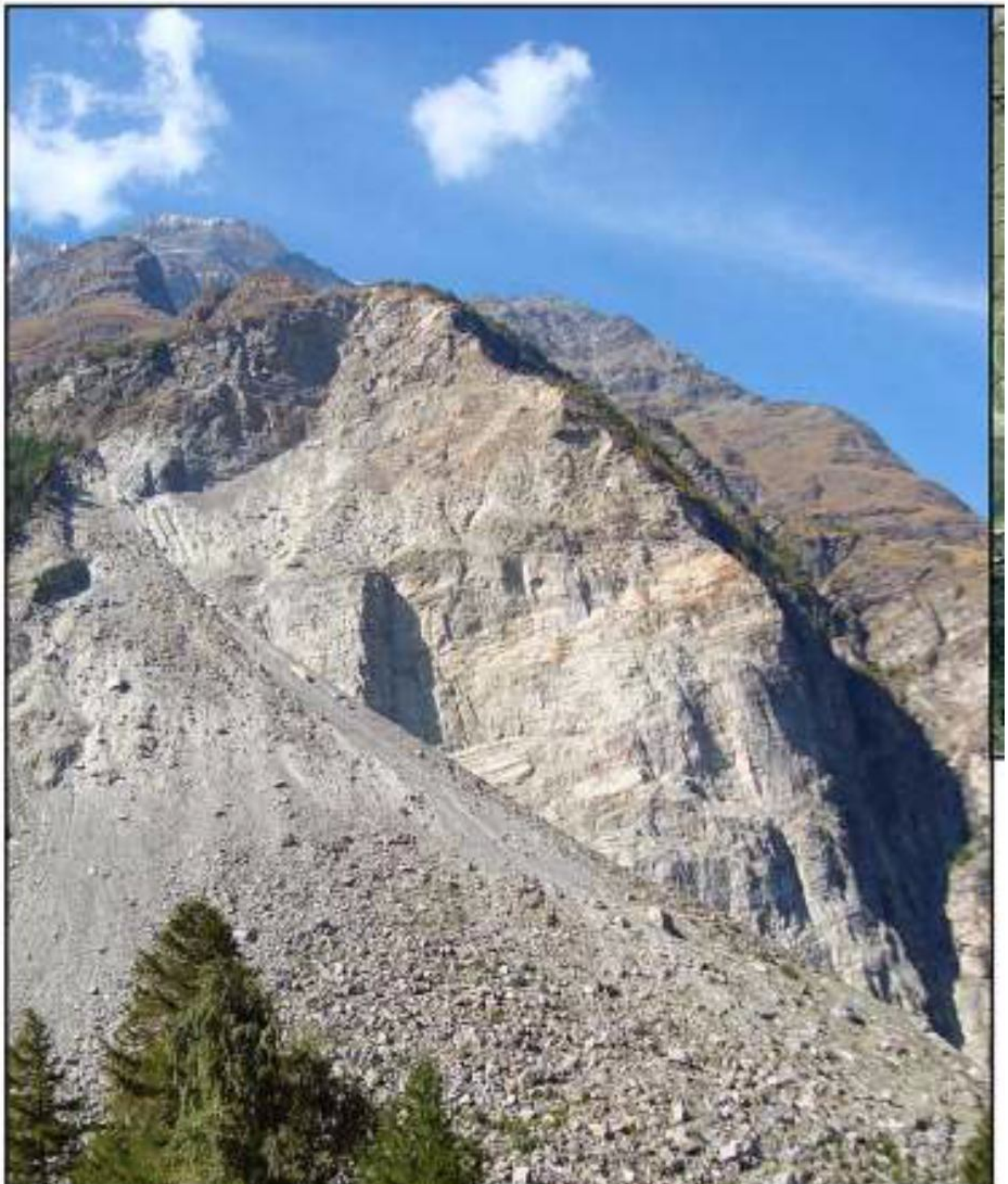




Composite rock slope kinematics at the current Randa instability, Switzerland, based on remote sensing and numerical modeling

V. Gischig[a], , F. Amann[a], J.R. Moore[a], S. Loew[a], H. Eisenbeiss[b], W. Stempfhuber[b], [1]

The current rock fall hazard is due largely to **secondary toppling** from the main scarp of a 30 million cubic meter rock fall from 1991. At present, there are 5 to 6 million cubic meters of unstable rock exposed in the inaccessible, 850 meter high scarp .



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Reliance on displacement monitoring for early warning and the difficulty in interpreting the data without a clear understanding of the underlying mechanisms has led to a situation where predictions are highly variable and generally unreliable.

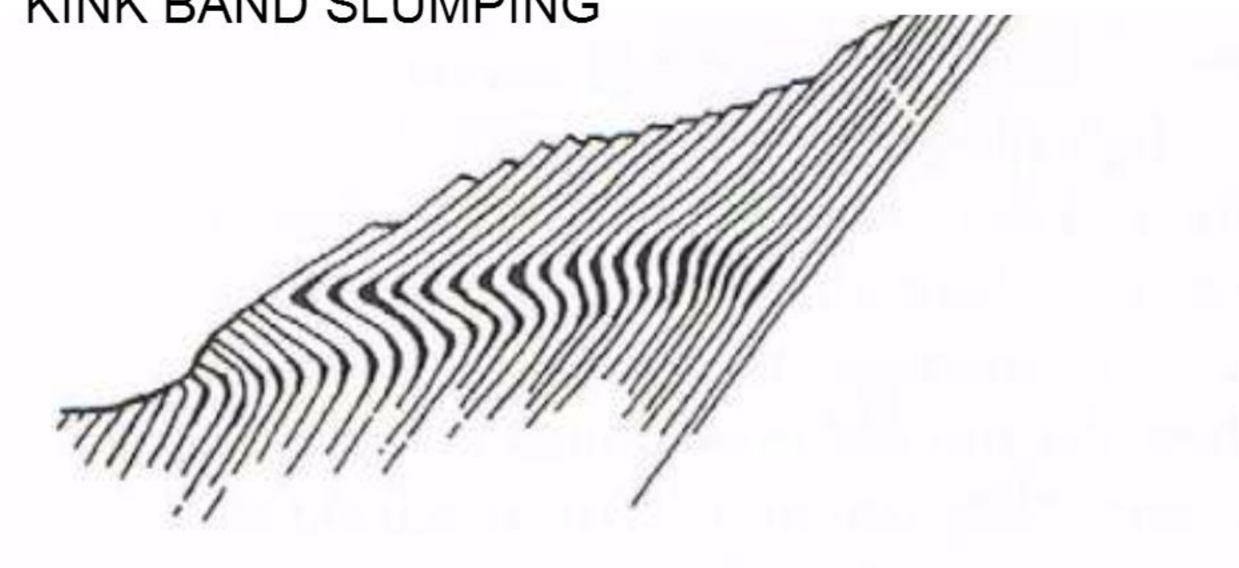
Erik Eberhardt

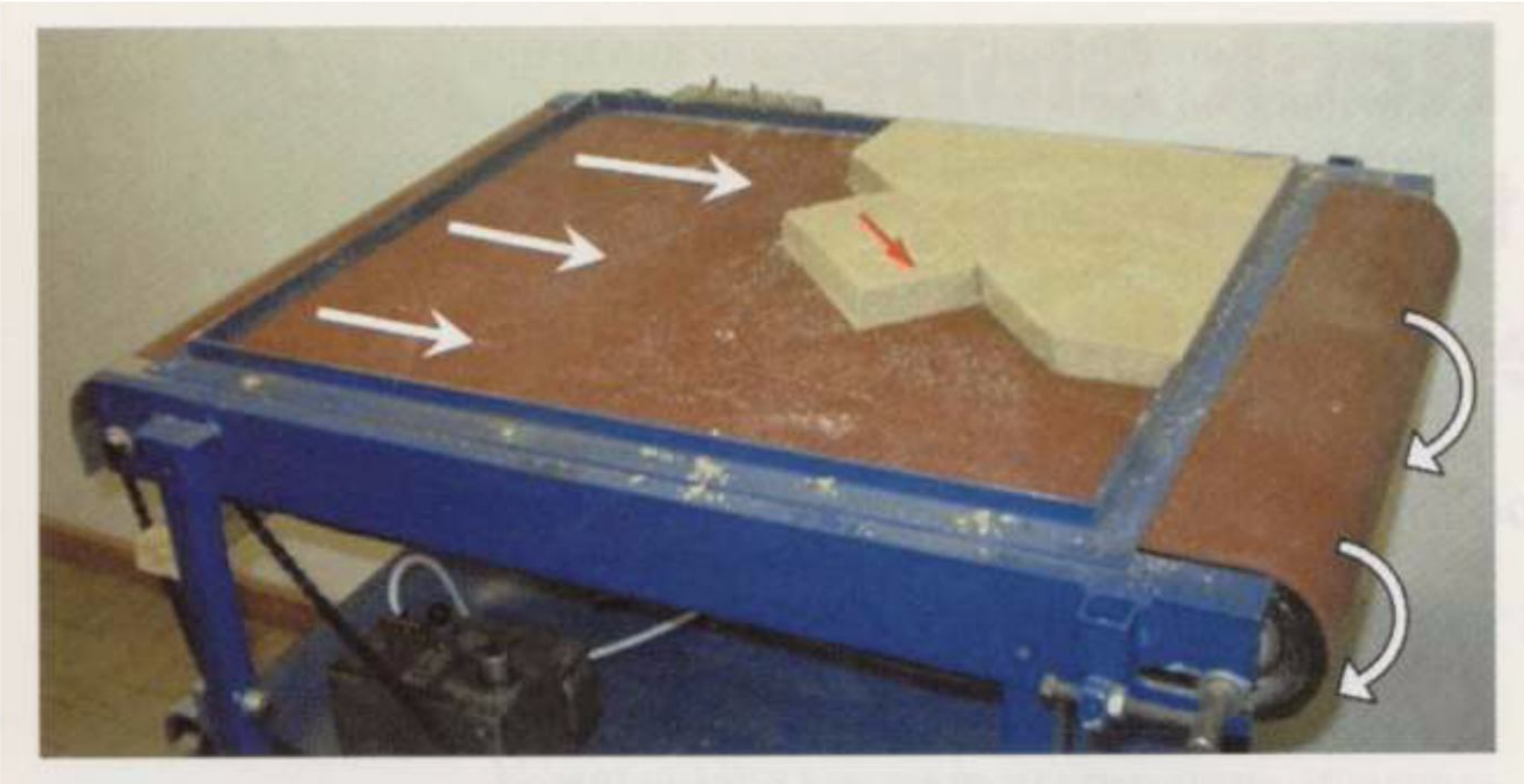
in *"The role of advanced numerical methods and geotechnical field measurements in understanding complex, deep-seated rock slope failure mechanisms - Canadian Geot. Journal, 2008 , pp 484-510*

BUT – Wyllie and Munn's handling of the Cardinal River Coal Mine toppling failure proves that reliable predictions can be relied on when the mode of failure <u>is well known.</u>

Some additional morphological and structural features of toppling rock masses.

KINK BAND SLUMPING



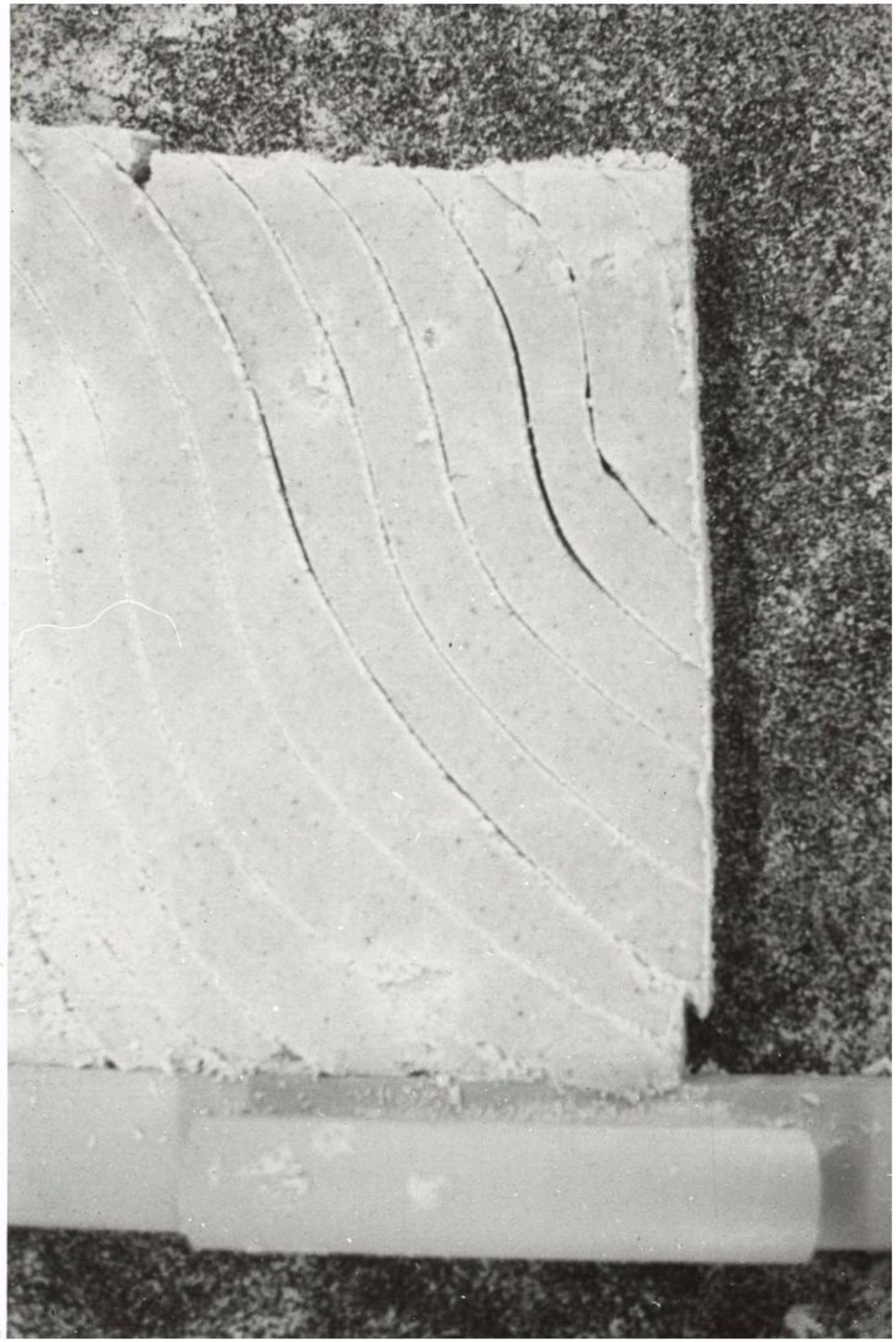


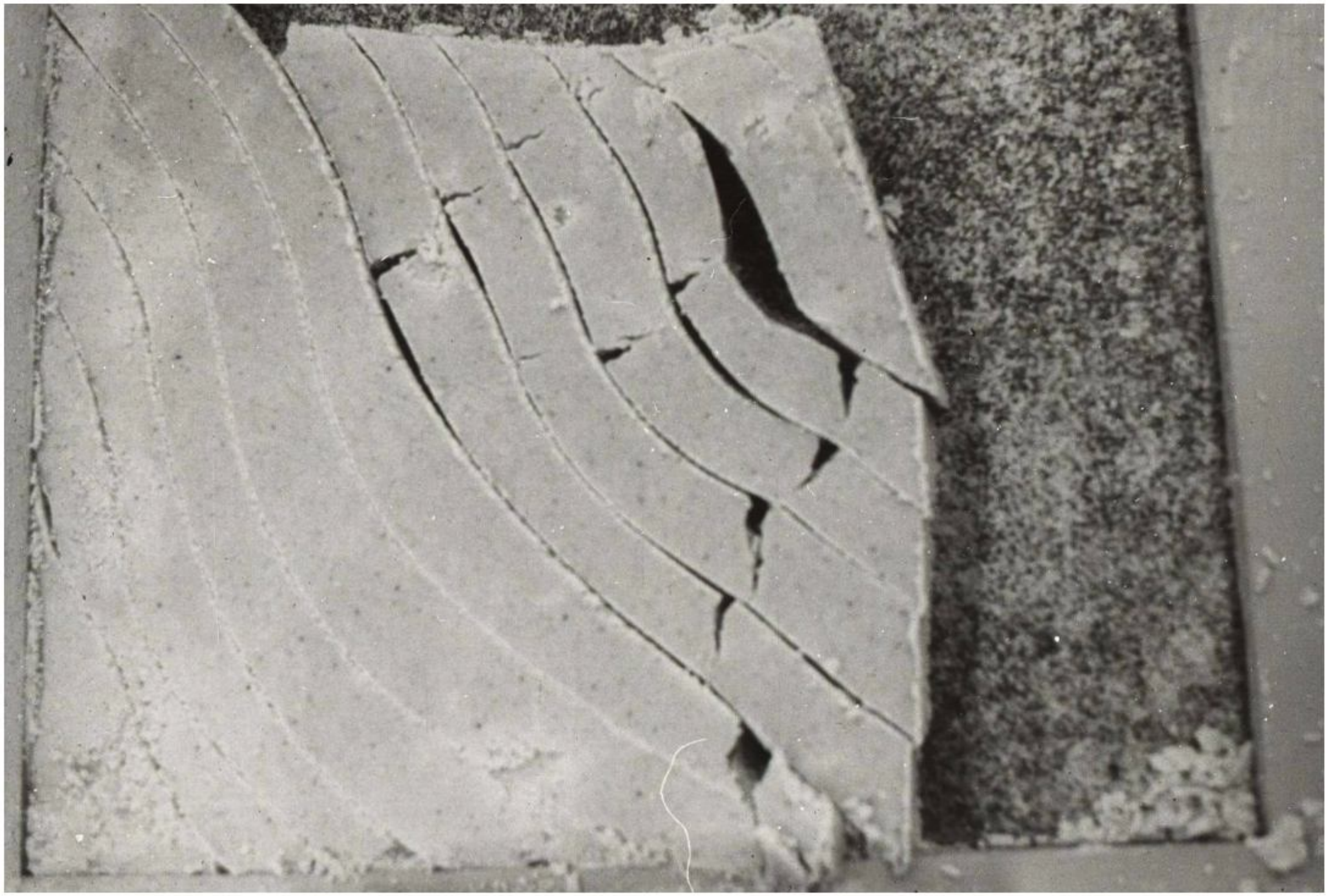
Base Friction Model

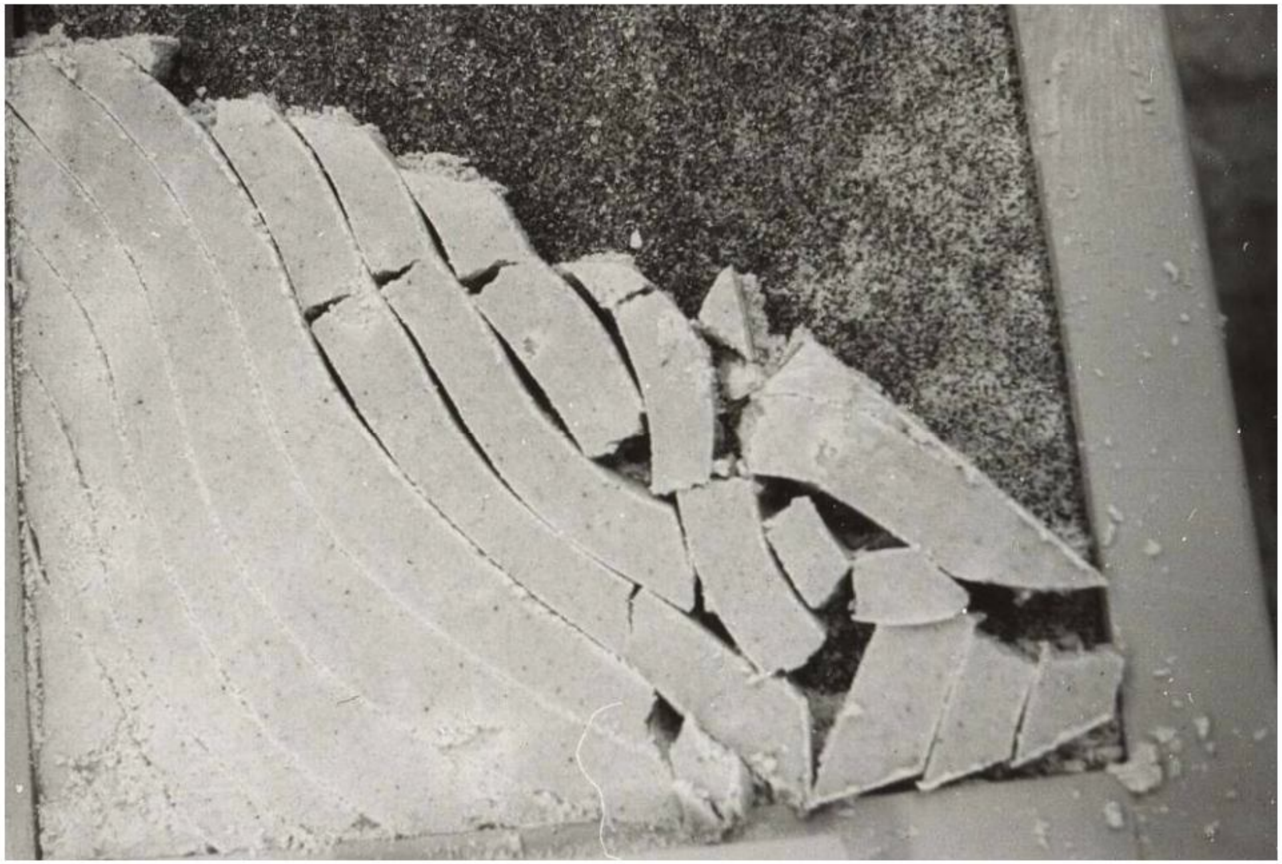
See: The theory of base friction models, by J. W, Bray and Richard E Goodman (1981) *International Journal of Rock Mechanics and Mining Sciences* (Pergamon) Vol. 18, no. 6., pp. 453-468

SLIDING-INDUCED TOPPLING - - Base Friction Model

Initiated by sliding on non-circular bedding







Toppling – induced Buckling

Toppling tends to produces three offset layers of differing rock-mass properties

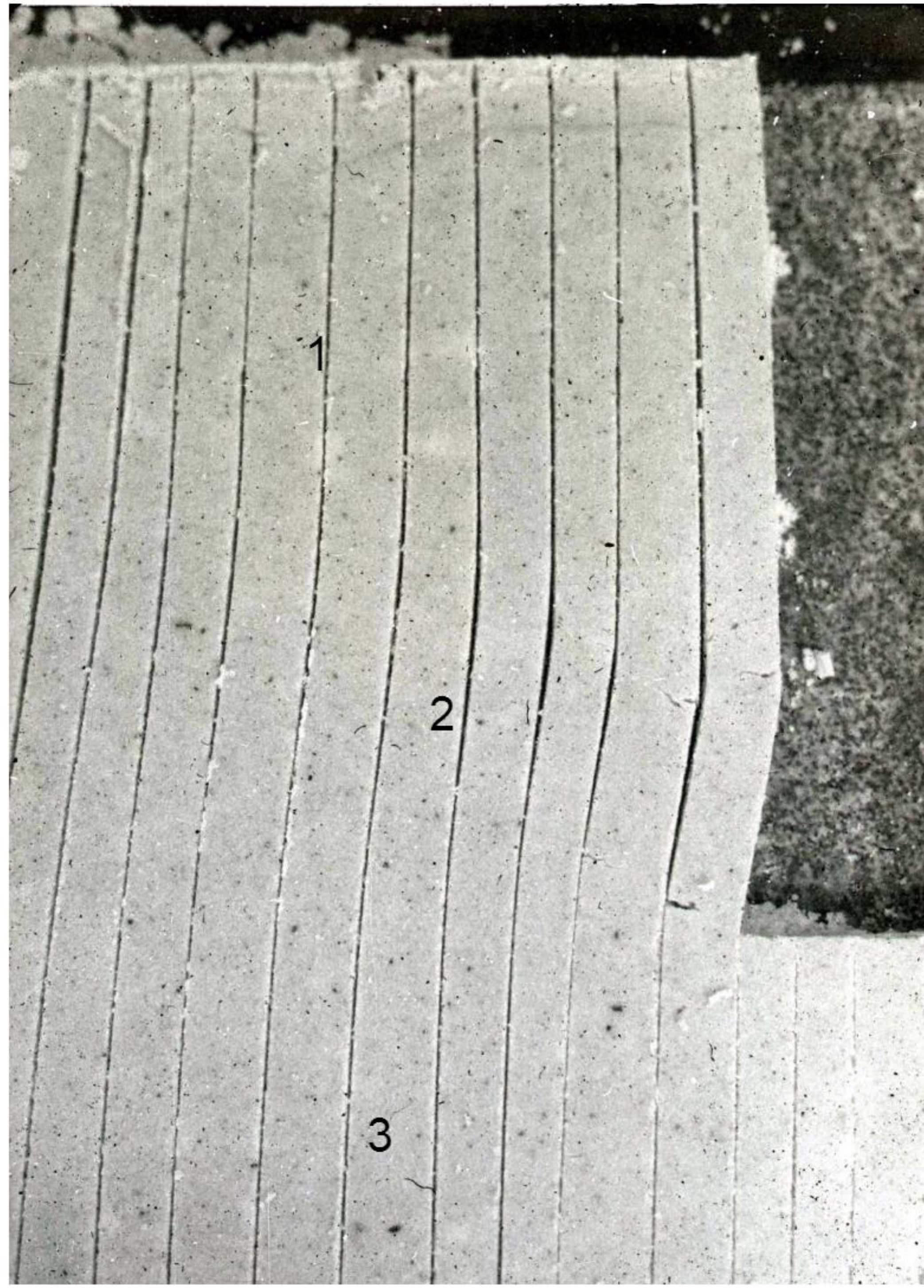
Here is a developing three layer case:

Stable lower zone, 3;

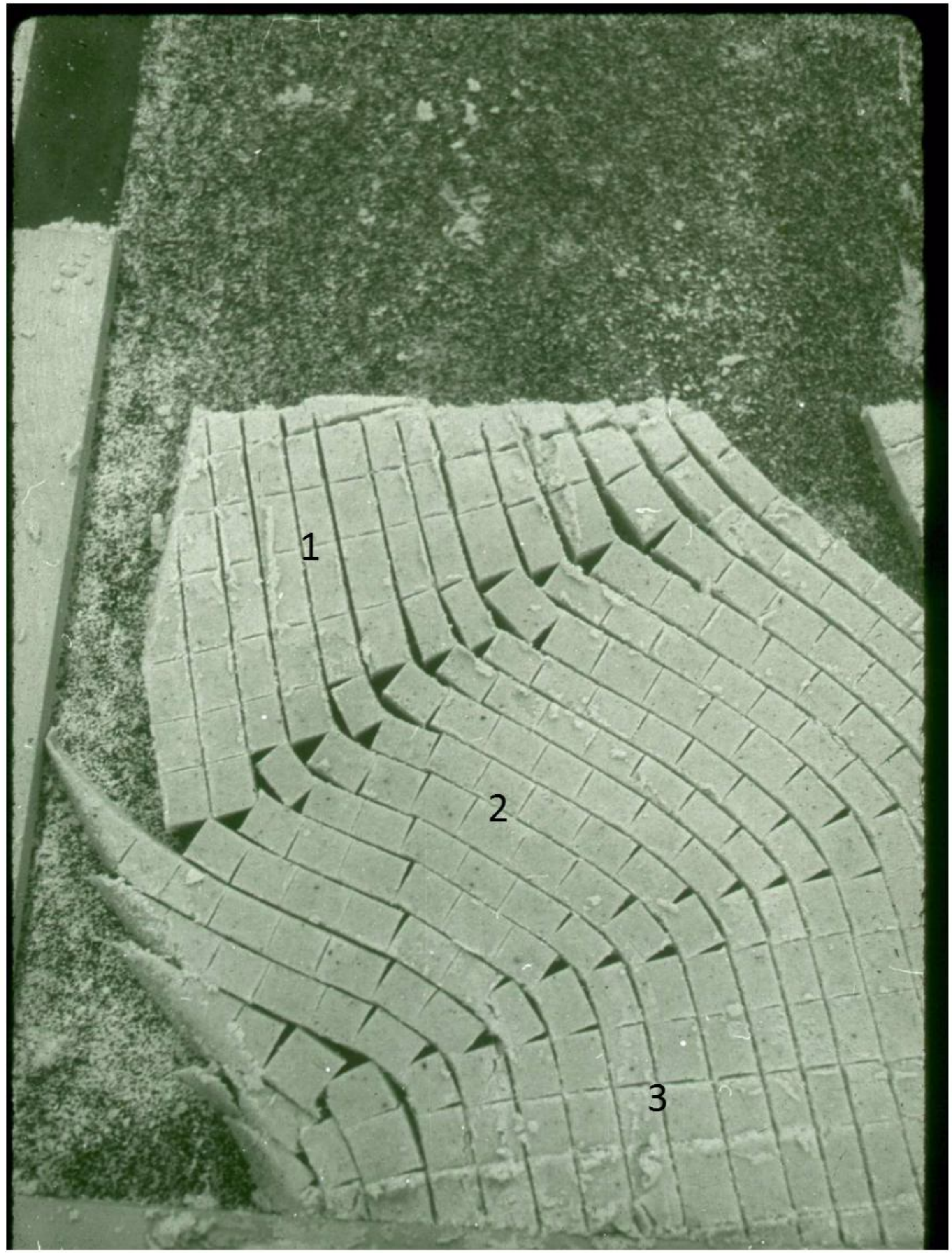
Toppling mid zone, 2;

Translating (rigid body motion) upper zone, 1.

This proceeds as in the next slide.



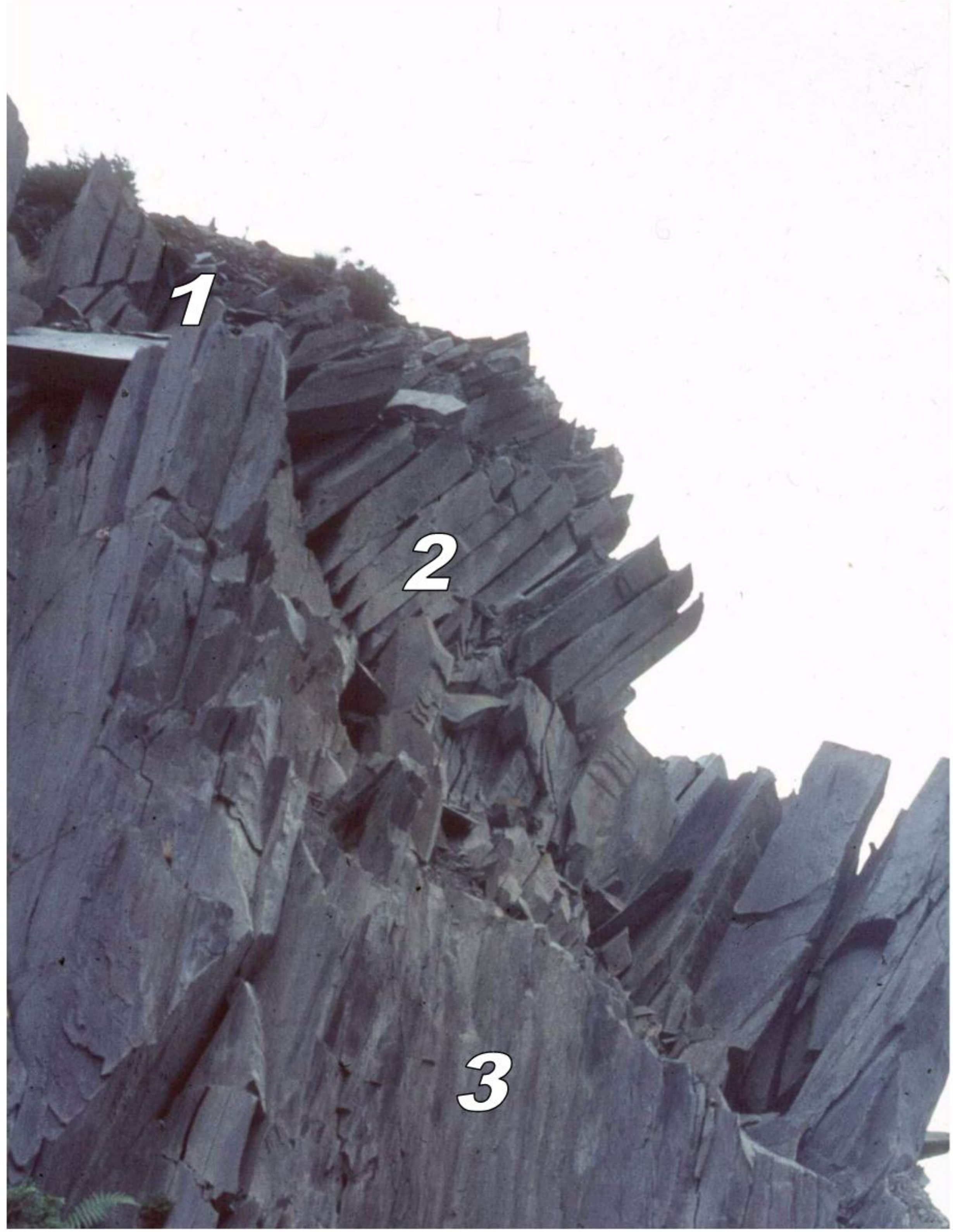
In this case, the upper layer (1) is a rather thick dead zone, but still more or less riding along on the toppling mass of the middle zone (2). The opening of the flexural cracks between the upper and middle (toppling) zone has reduced the shear stress required to sustain active flexure in the layers of the upper zone..



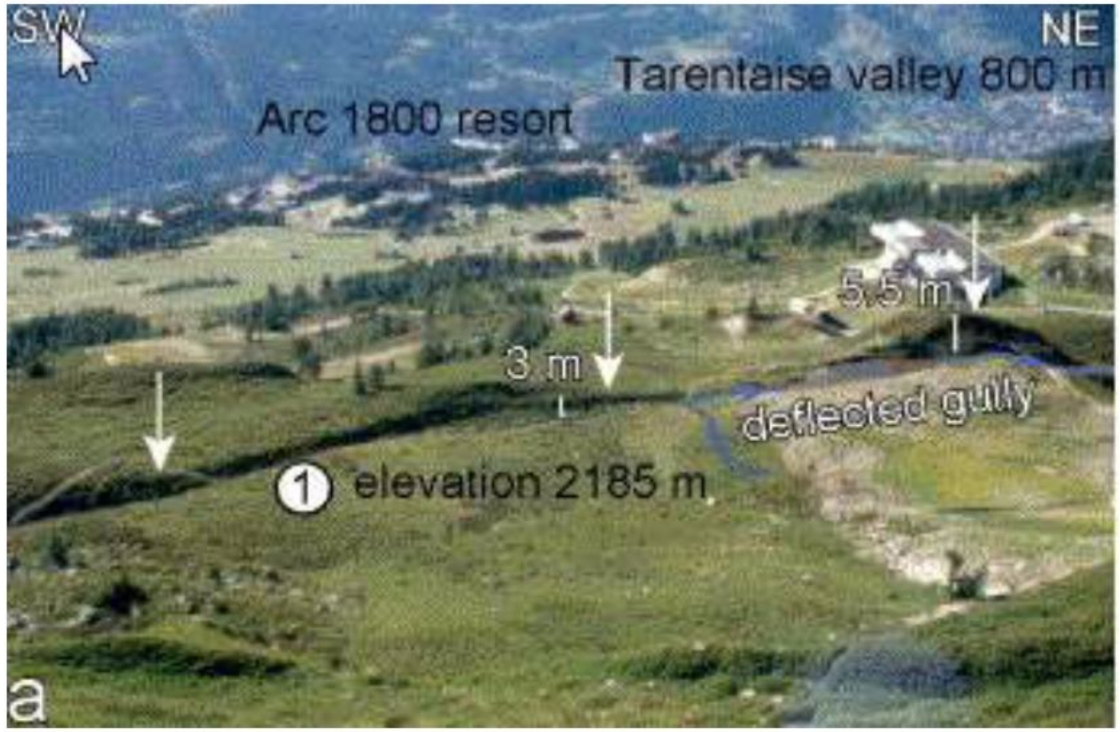
A toppling slope in North Devon, England.

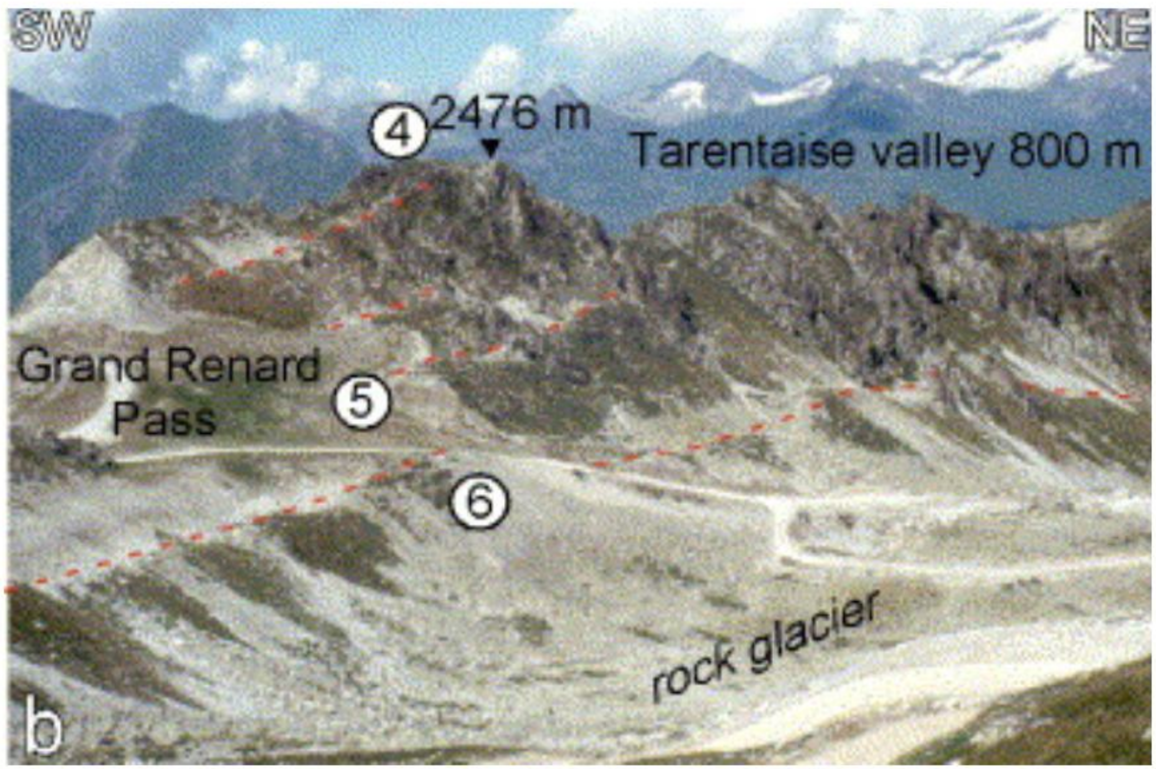
The three zones are numbered.

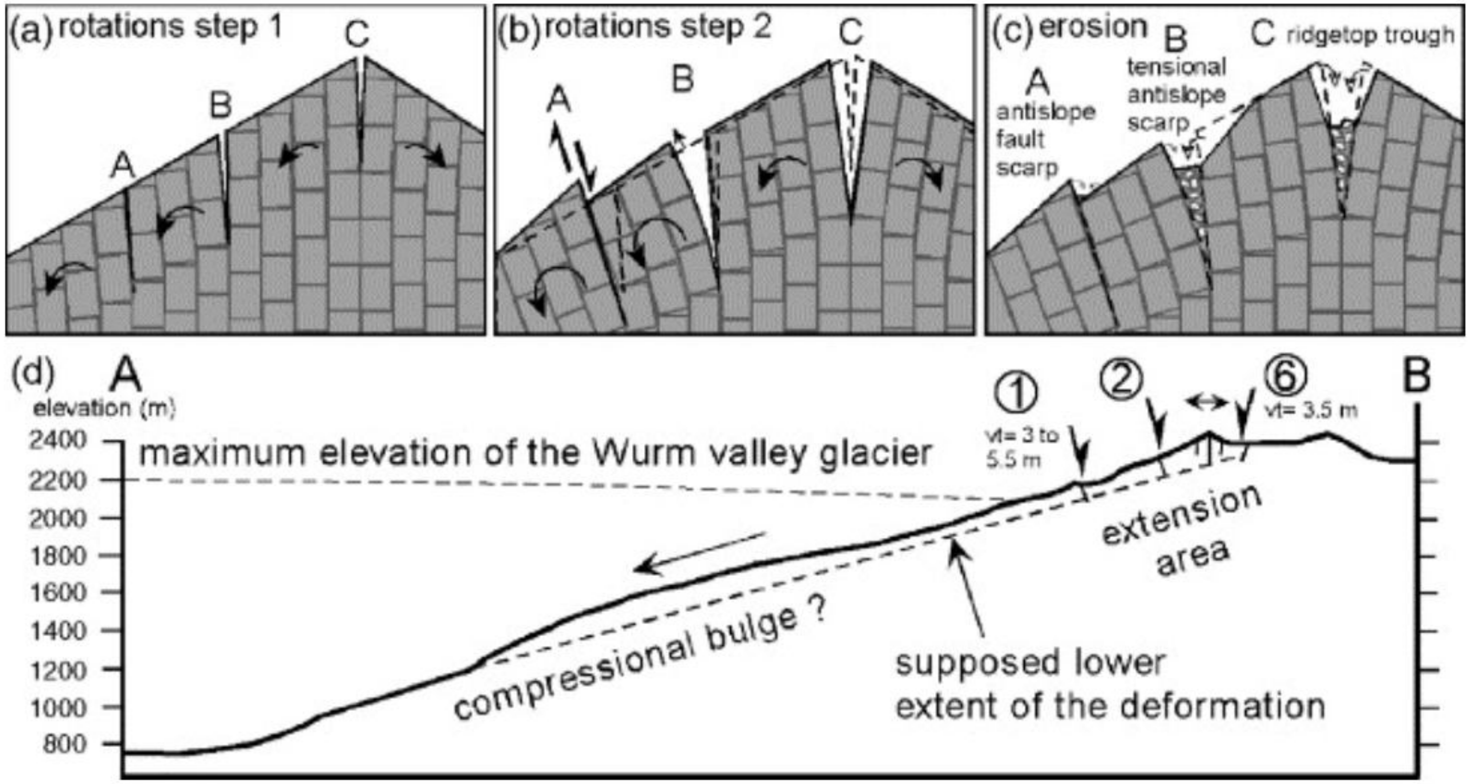
Notice the edge to plane contacts beneath the figure 2, as can be seen in the base friction models.



Two of many obsequent scarps mapped in the French Alps near Les Arcs ski resort; (Hippolyte, Brocard, Tardy, Nicoud, Bourles, Braucher, Menard, and Souffache **Tectonophysics, 418, no 3 (2011)**







Model of gravitational sackung deformation at Les Arcs. (a, b, c) observed in the Aiguille Grive massif. The ridge top troughs (C) are tension cracks resulting from toppling of vertical rock layers in the two slope directions. The tensional antislope (obsequent) scarp (B) results from toppling of the rock layers and erosion mainly of the steep uphill face (Jahn, 1964). (d) Topographic cross-section of the Aiguille Grive massif (location in Fig. 2) with location of the deformation. The valley glacier was settled on the Carboniferous silts and sandstones

Spread-sheet analysis of block toppling:

Goodman and Bray (1976) provided a fundamental kinematic basis and method for one particularly relevant block toppling geometry - - *toppling on a stepped base.*

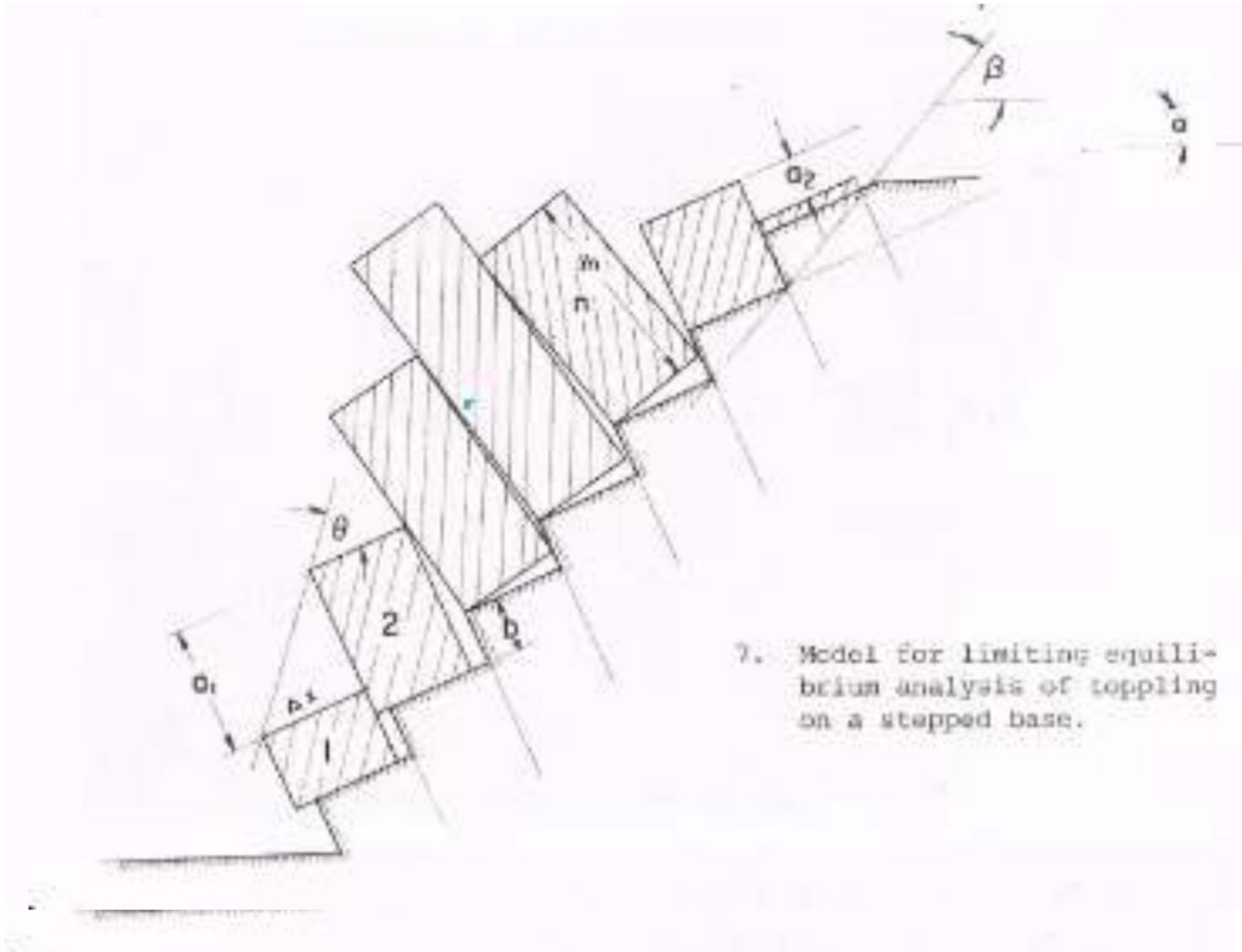
Begin with the top-most toppleable block, calculating the larger of the reactions from the next lower block required for equilibrium by (1) toppling or (2) sliding,

Transfer the higher of these as load onto the next lower block, etc one by one through all blocks to the lowest block.

The minimum value of P for the lowest block's critical mode measures the safety of the whole mass of blocks.

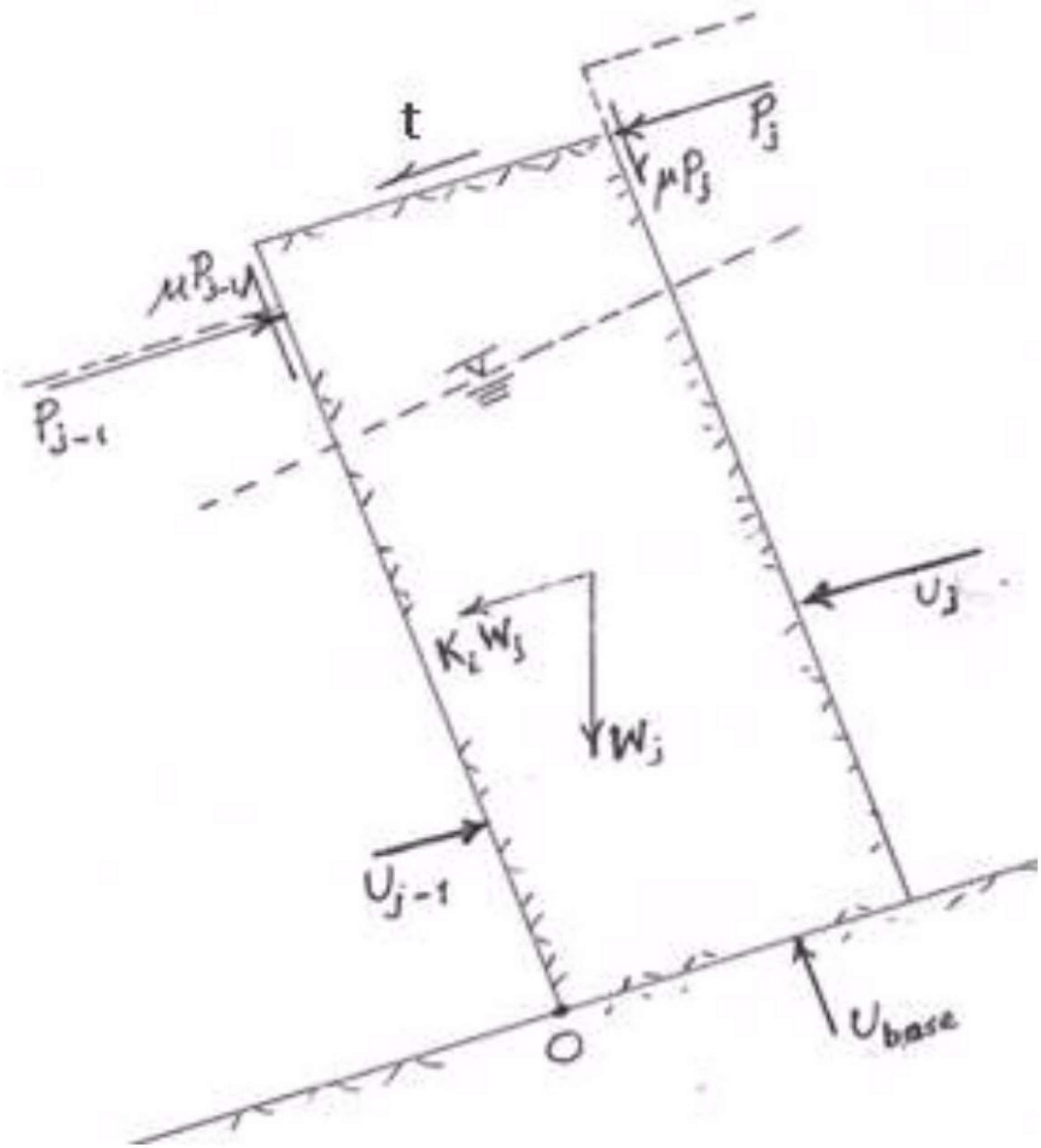
The process can be tedious for general design analysis & C*are is needed to avoid errors.*

But it is practical, and general, and is easily customized for particular conditions and local loadings, e.g. earth pressure, soil creep, locally changed properties.



7. Model for limiting equilibrium analysis of toppling on a stepped base.

Goodman and Bray Spread Sheet 1977 Analysis for Toppling and Sliding on a Stepped Base


t
P_j
μP_j
μP_{j-1}
P_{j-1}
U_j
$K_i W_j$
W_j
U_{j-1}
O
U_{base}

SOME DEVELOPMENTS BY LATER AUTHORS for Block Toppling:

- **Zanbak (1983) works the spread sheet to provide some solution charts.**

- **Scavia, Barla & Bernardo (1990) produce a probabilistic approach.**

- **Ke, Thapa, and Goodman (1994) add soil creep over the toppling slope to the spread sheet and compare results with DDA modeling using the friction reduction method .**

- **Bobet (1999) gives an efficient closed form solution to a special case.**

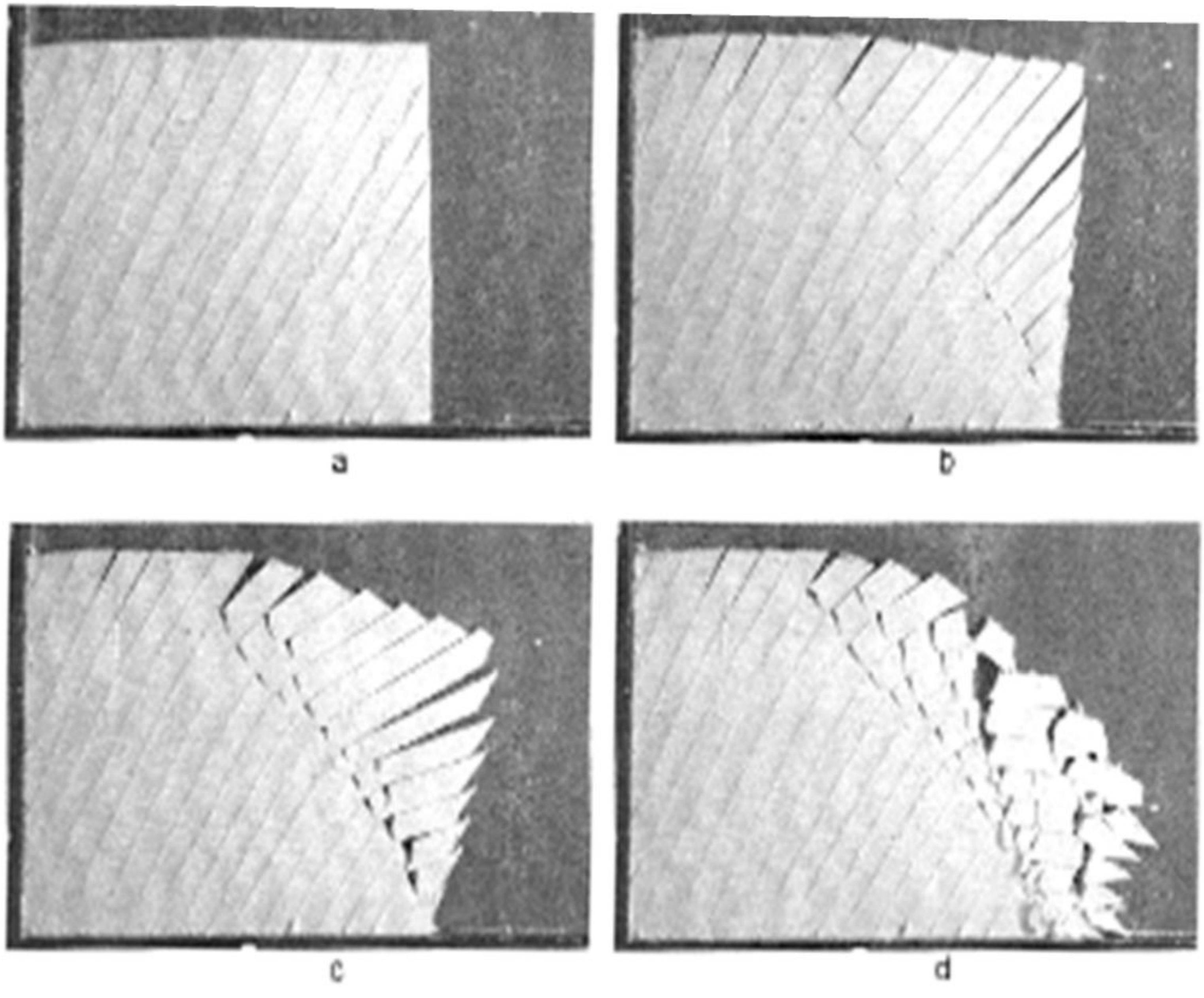
- **Sagaseta et al (2001) integrate ordinary differential equations for toppling derived by shrinking the blocks to infinitesimal thickness.**

- **Liu, Jaksa & Meyers (2009) improve spread sheet computational efficiency**

- **Tatone and Grasselli (2010) introduce Monte Carlo methods into the spread sheet approach producing the probabilistic program ROCKTOPPLE.**

- **Amini, Majdi, and Veshadi (2012) – discuss analysis for block-flexure toppling**

- **Rock Science company releases commercial code: *Rock Topple* (2013)**



FLEXURAL TOPPLING - base friction model example

Structural Analysis of FLEXURAL TOPPLING

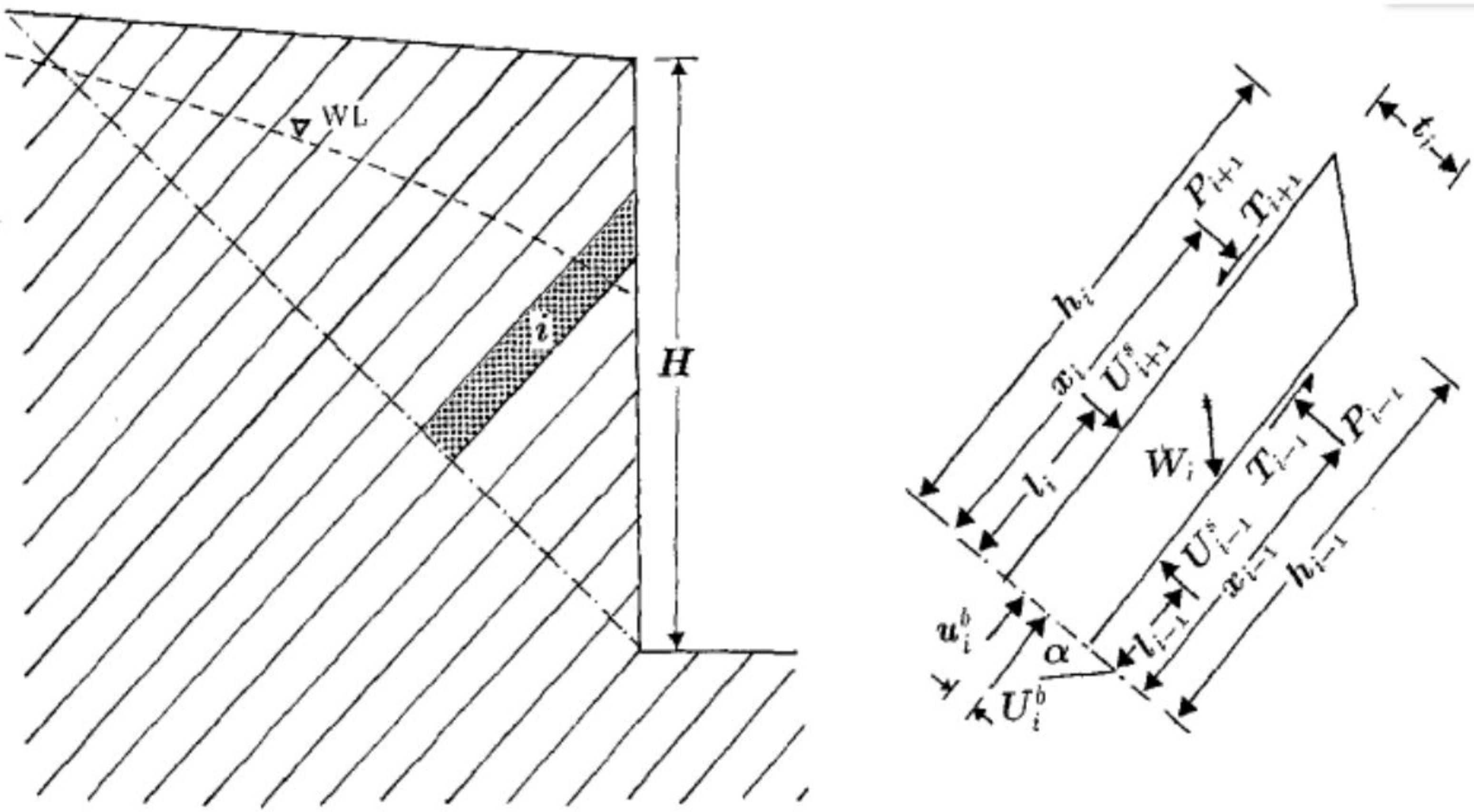
Aydan & Kawamoto 1972- Developed simplifed limit equil analysis from Base friction tests and column theory,.. **An Epic Paper**

Adikhary et .al, 1997 - Improved the above for slopes based on meticulous centrifuge model tests and generalized results with dimensionless design charts.

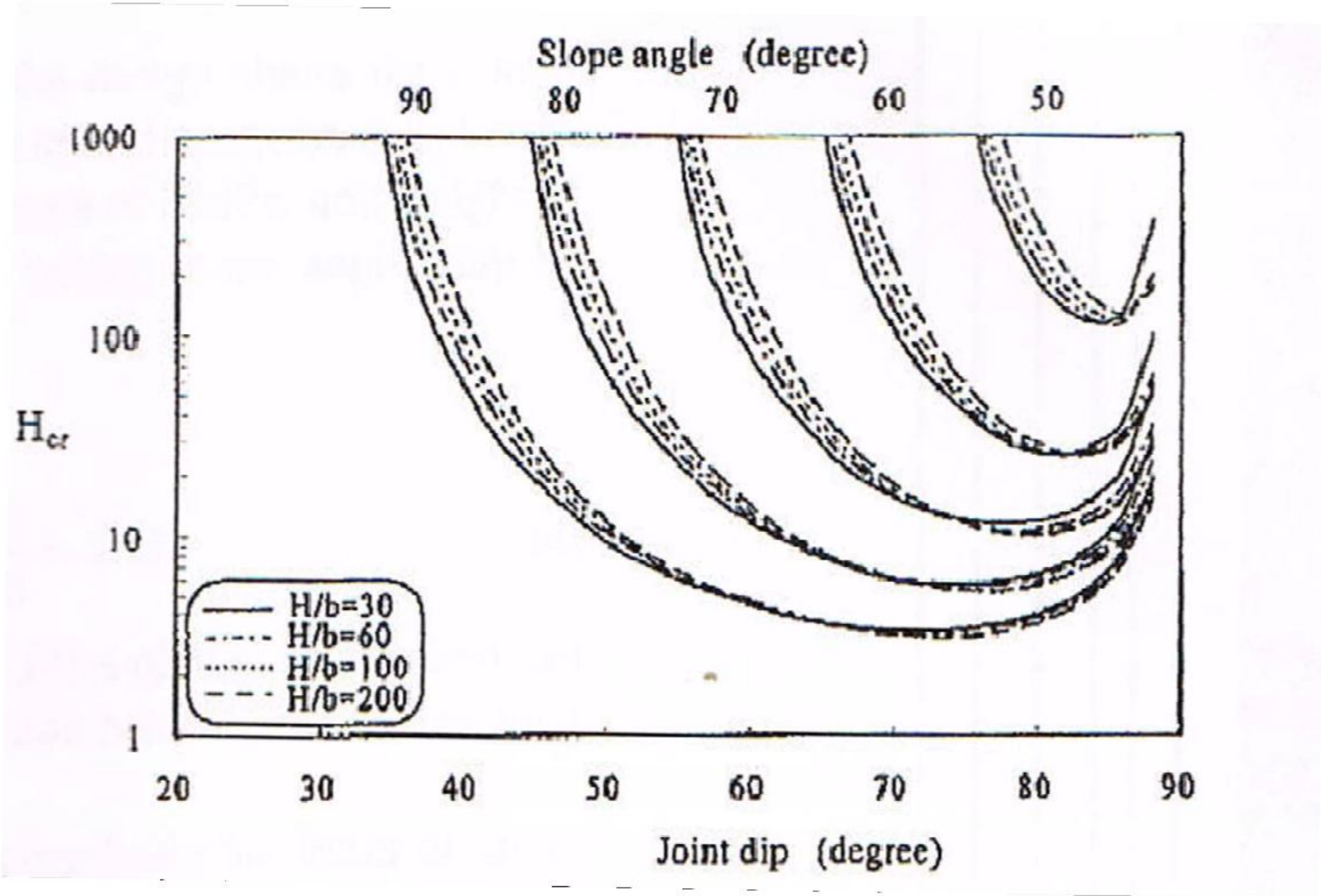
Adikhary & Dyskin, 2006), Applied Cosserat plasticity to the problem

Amini, Majdi and Ayden (2009) – Discussed design applications in slopes and tunnels

[1] With kind permission from Springer Science+Business Media B.V..



Aydan & Kawamoto's freebody diagram for limit equilibrium analysis of a flexural-toppling-slope. [1]



Flexural Toppling Design Chart for ϕ = 30°, one of 7 such design charts produced by Adikhary, Dyskin, Jewell, and Stewart (1997);

H_{cr} = $[\gamma n\ H^2] / [(\sigma_t/F)\ b]$, in which: **H_{cr}** = a dimensionless critical slope number; γ = unit weight of the rock; σ_t is the modulus of rupture (tensile strength in flexure) of the rock; **H** is the slope height; **F** is the load factor of safety; **b** is the column thickness (= spacing of column-defining joints or beds); and **n** is a weighting factor to facilitate plotting an iterative solution by arbitrarily increasing the unit weight.

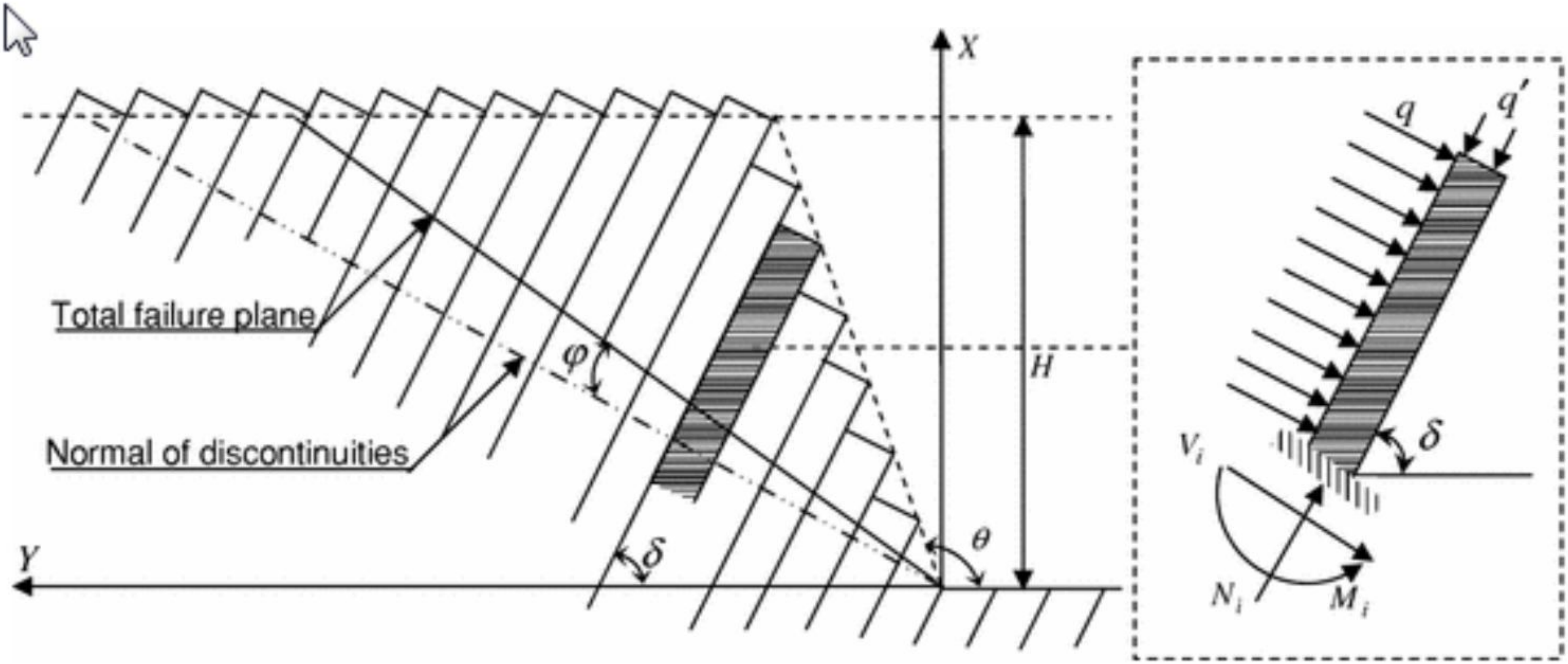
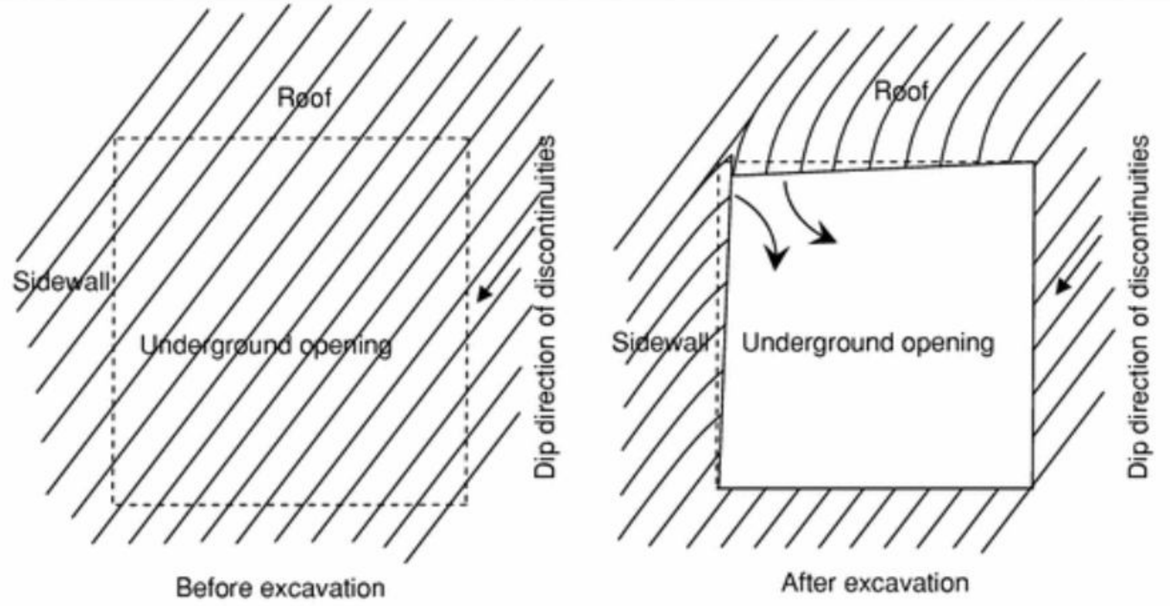


Fig. 9 Rock slope with a potential of flexural toppling failure

From M. Amini, A. Majdi, and O Ayden (2009) Stability analysis and stabilization of flexural toppling failure, *Rock Mechanics, Vol 42, pp. 751-782*

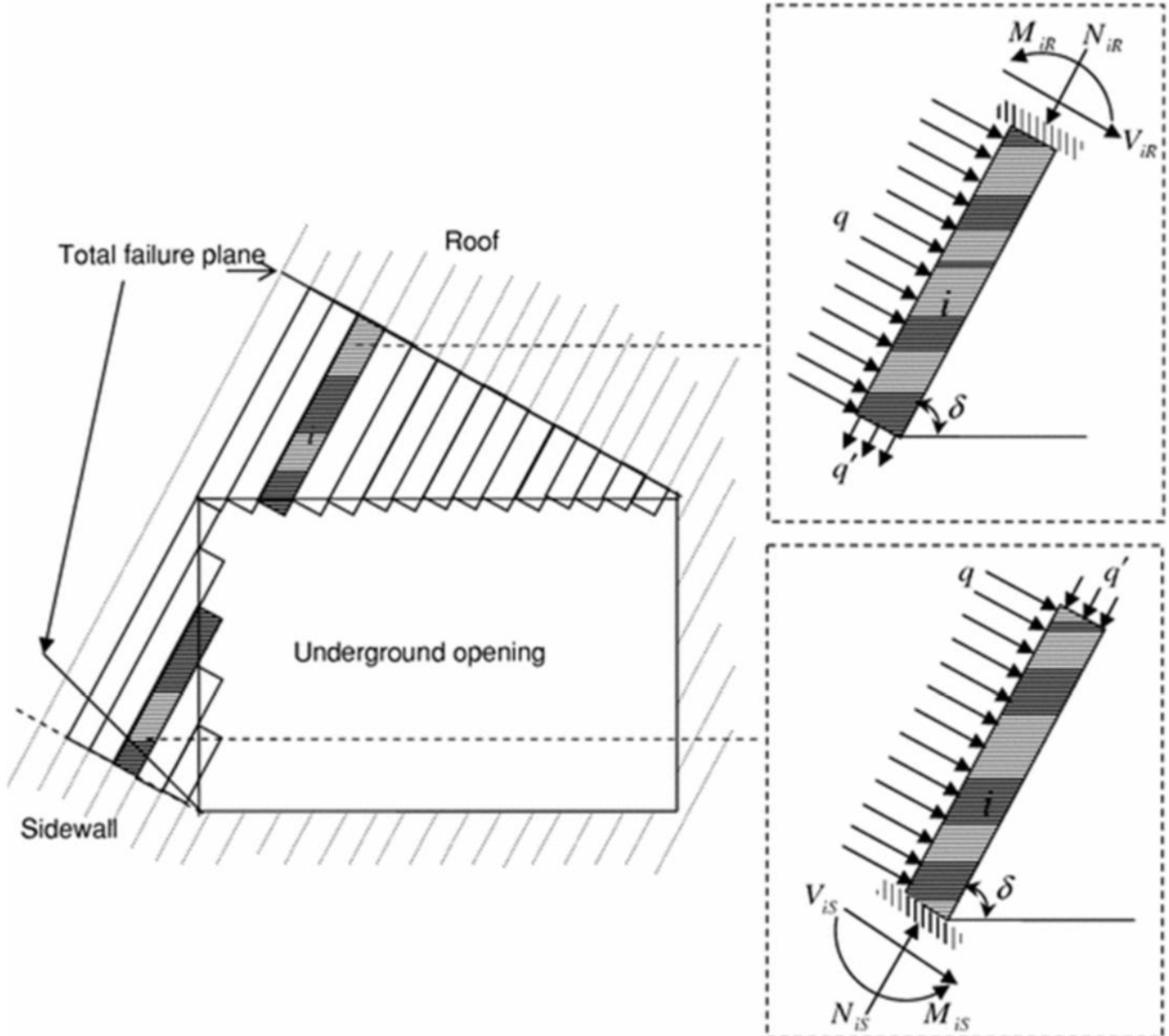
Flexural toppling in underground openings -- Actual case study (the Chaloos tunnel, Iran)



a

b

From M. Amini, A. Majdi, and O Ayden (2009) Stability analysis and stabillization of flexural toppling failure, *Rock Mechanics, Vol 42, pp. 751-782*



From M. Amini, A. Majdi, and O Ayden (2009) Stability analysis and stabilization of flexural toppling failure, ***Rock Mechanics, Vol 42, pp. 751-782***

PHYSICAL MODELS

<u>Tilt Models</u> – (Hoffmann – 1972)

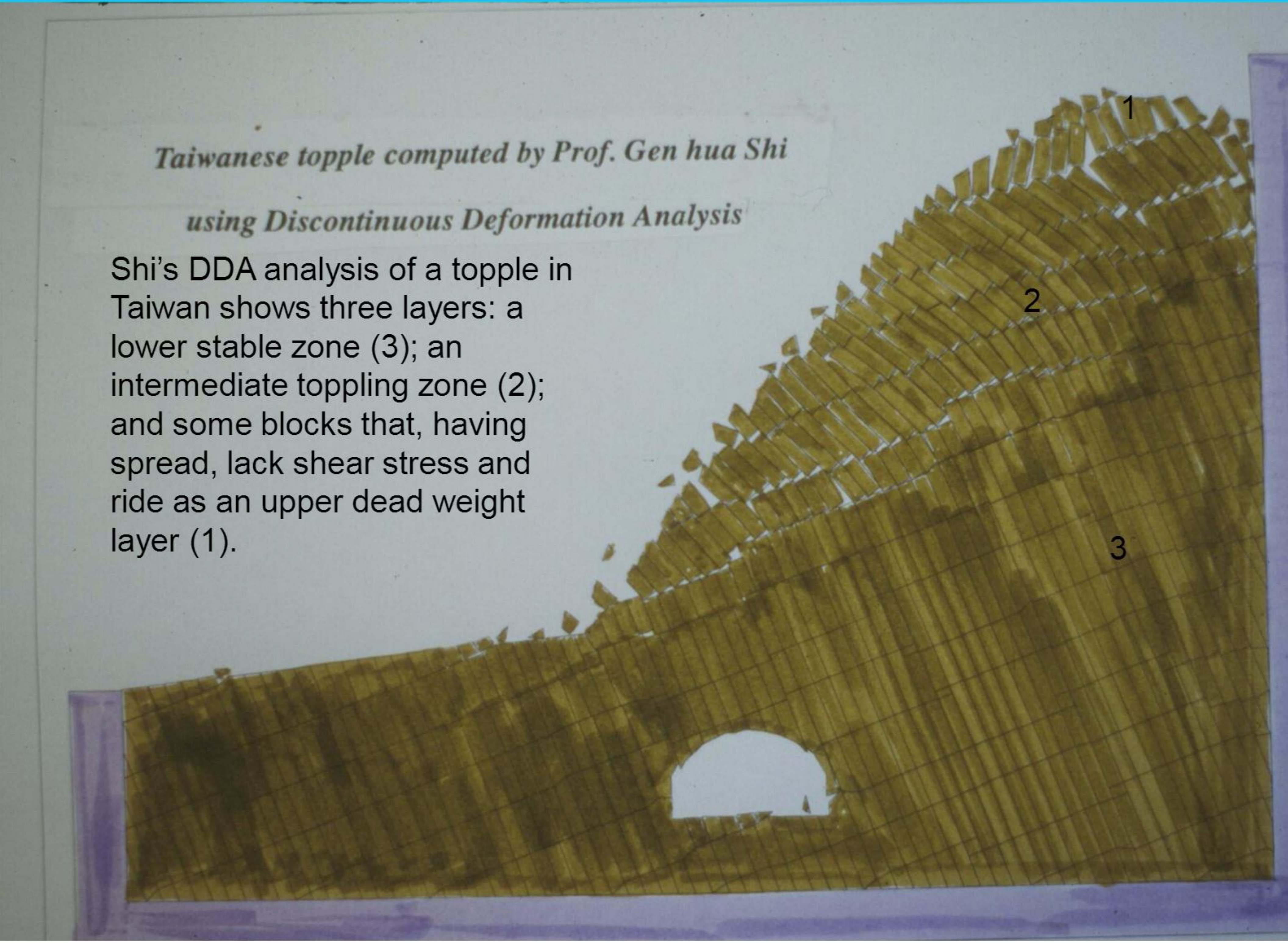
<u>Base friction model studies</u> (Bray & Goodman, 1981)

<u>Centrifuge models</u> (Chen,Zhang, Wang, Xing, 2006); (Adikhary et al 1997)

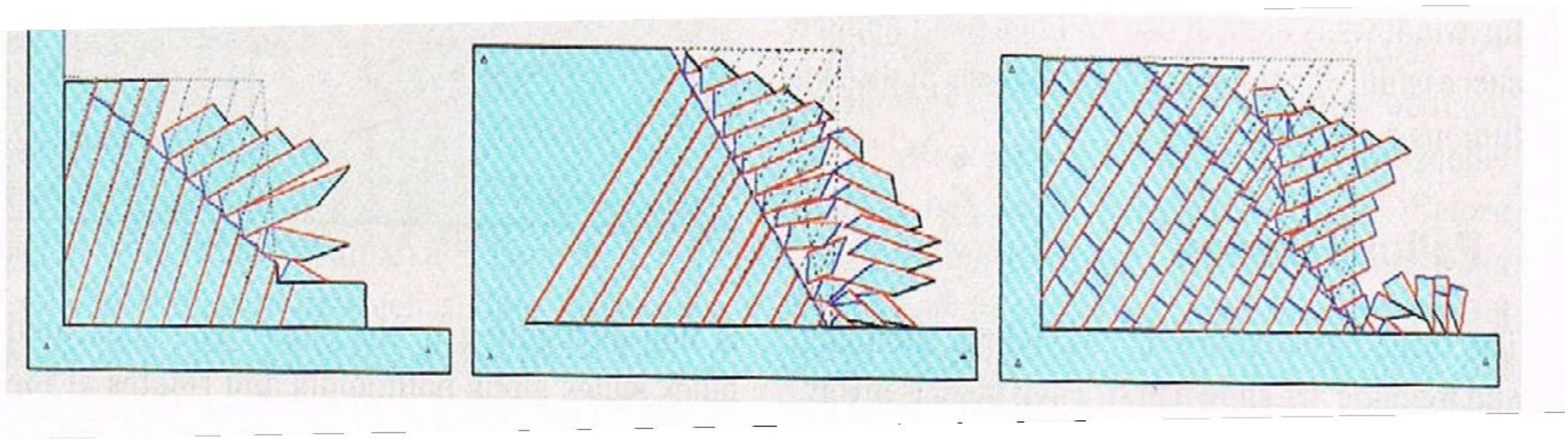
NUMERICAL MODELS - - many examples in the literature

DDA (Shi, G.H.)

and UDEC (Alzo'ubi, Martin & Cruden 2010)



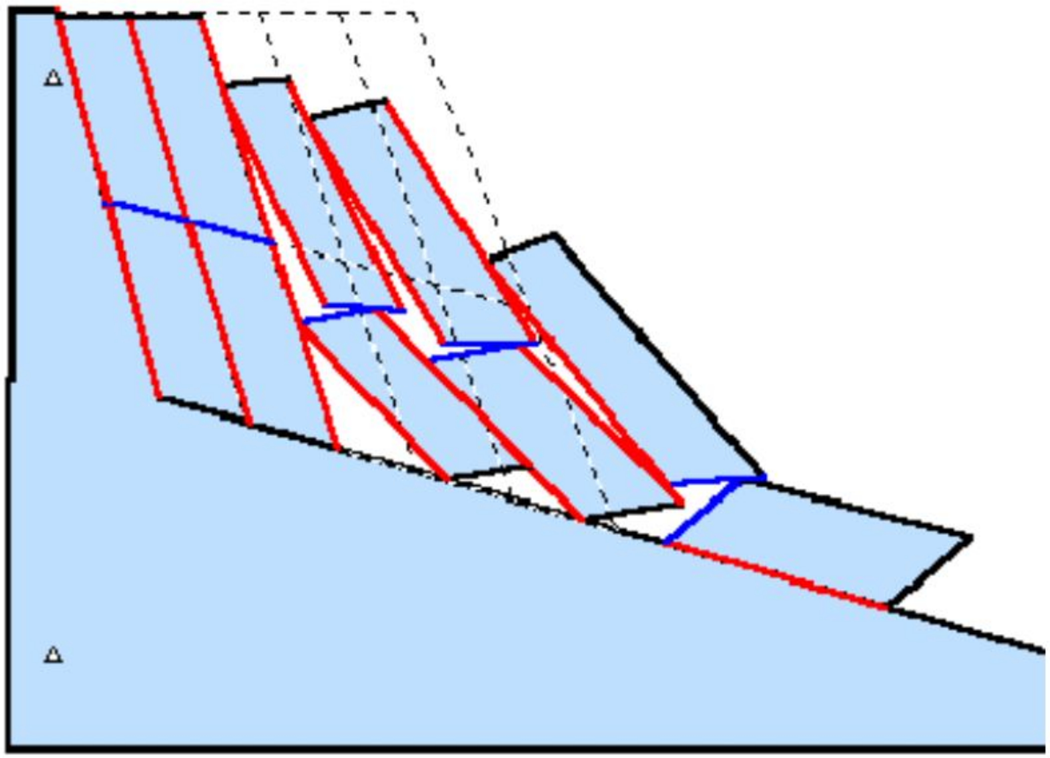
Shi's DDA analysis of a topple in Taiwan shows three layers: a lower stable zone (3); an intermediate toppling zone (2); and some blocks that, having spread, lack shear stress and ride as an upper dead weight layer (1).

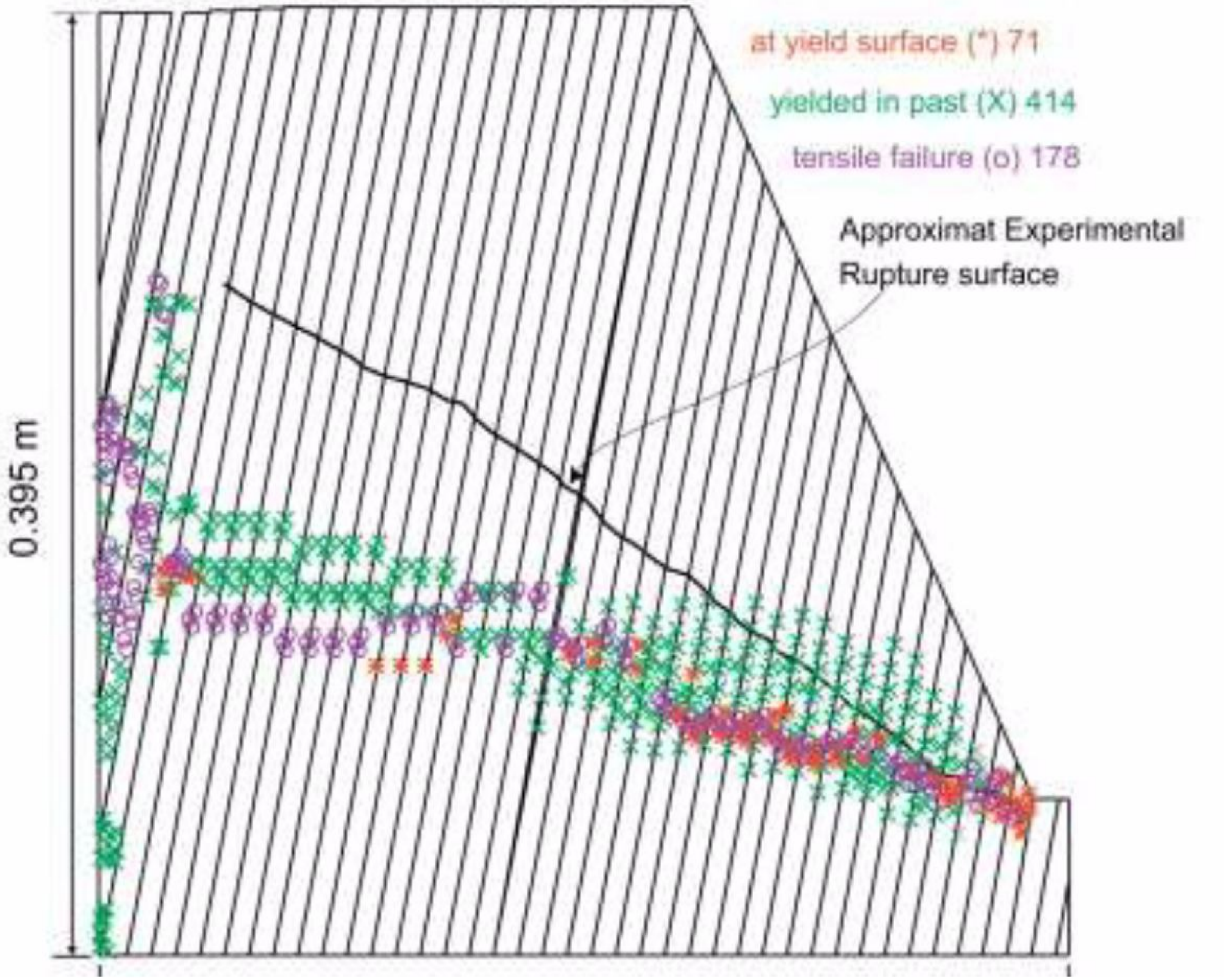


Discontinuous Deformation Models (DDA) of toppling rock slopes from Goricki and Goodman (2003)

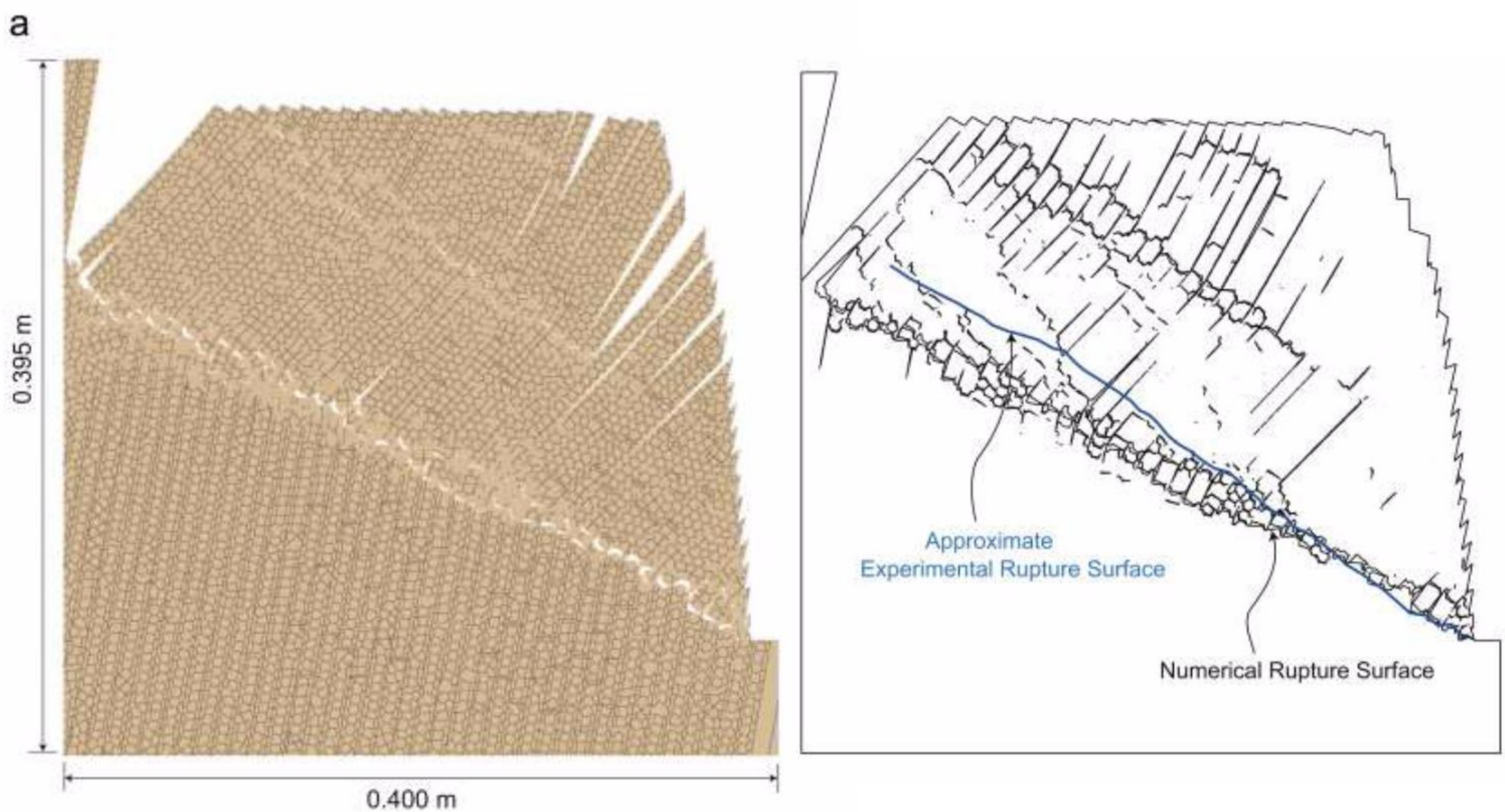
Comparison of Base Friction and DDA

From Thesis of Andrew Goricki, Graz Inst. Tech.





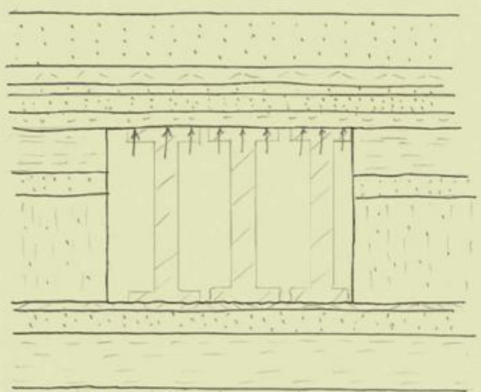
Conventional UDEC modelling results of Alzo'ubi, Martin & Cruden (2010) compared with actual flexural toppling rupture surface in **centrifuge models** by Adikhary et al,1997.



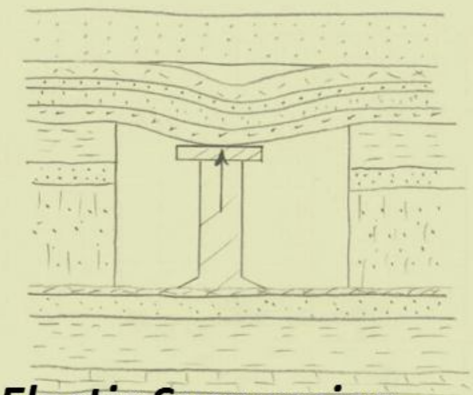
Improved numerical simulation of Adikhary's centriguge model of flexural toppling by Alzo'ubi, Martin and Cruden (2010) using the UDEC damage model with internal flaws within the inclined columns ("Voronoi joints" of Lorig and Cundall (1987))

A CONTINUUM OF DESIGN-APPROACHES FOR EXCAVATING SAFE SLOPES IN POTENTIALLY TOPPLING FORMATIONS

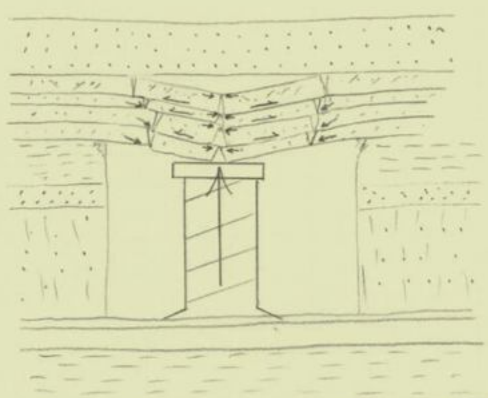
A Continuum of 4 design philosophies – in order of increasing support-force and rock deformation for a hypothetical roof.



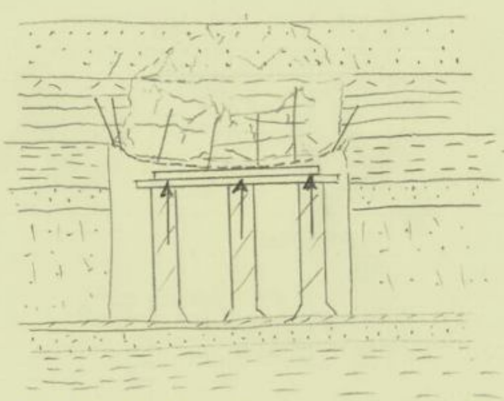
1) ***Underpinning support*** – no deformation allowed - - Roof doesn't know about excavation



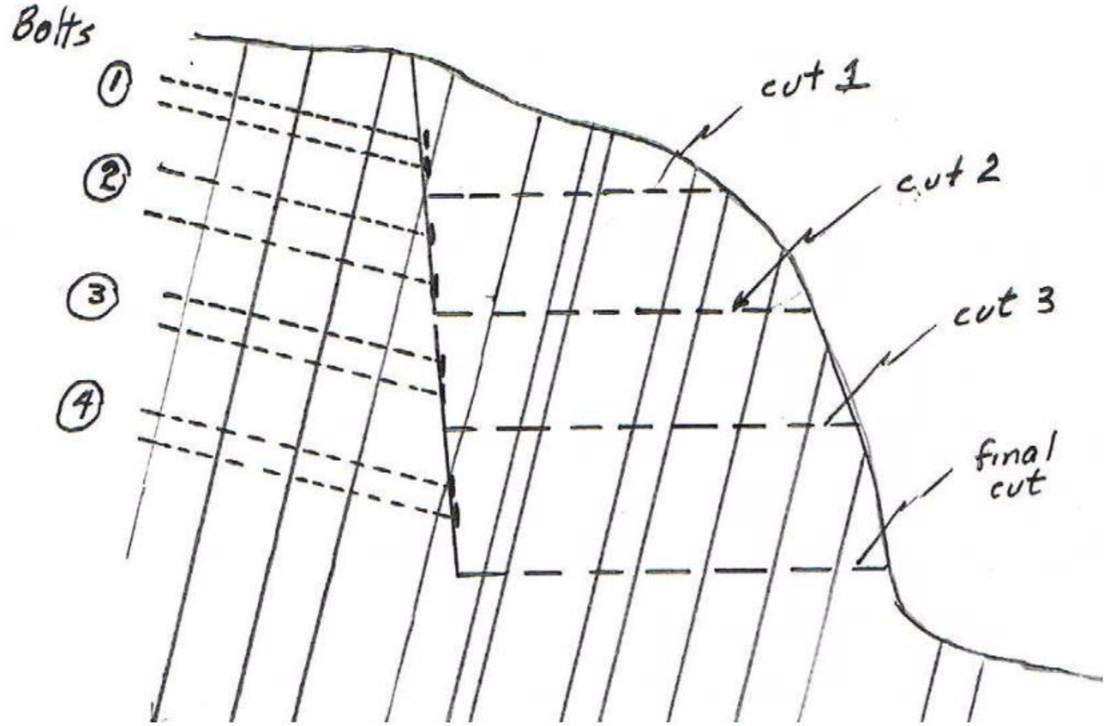
2) ***Elastic Suspension*** – stresses below elastic limit



3) ***Frictional Suspension*** - Roof naturally mobilizes frictional resistance --- "**PLASTIC DESIGN**"



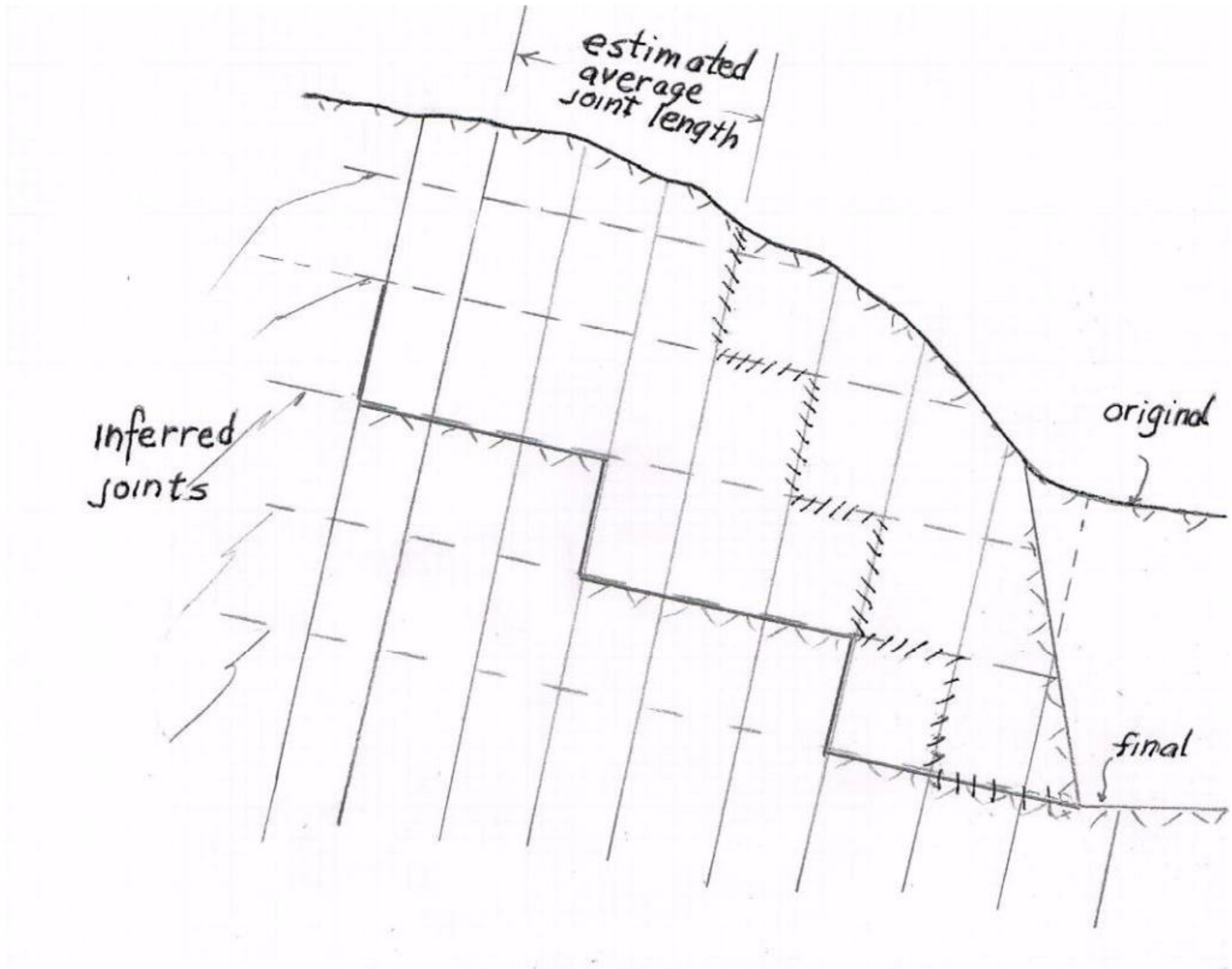
4) ***Dead Weight Support*** - Roof collapse is prevented by supporting the total weight of potential fall-out.



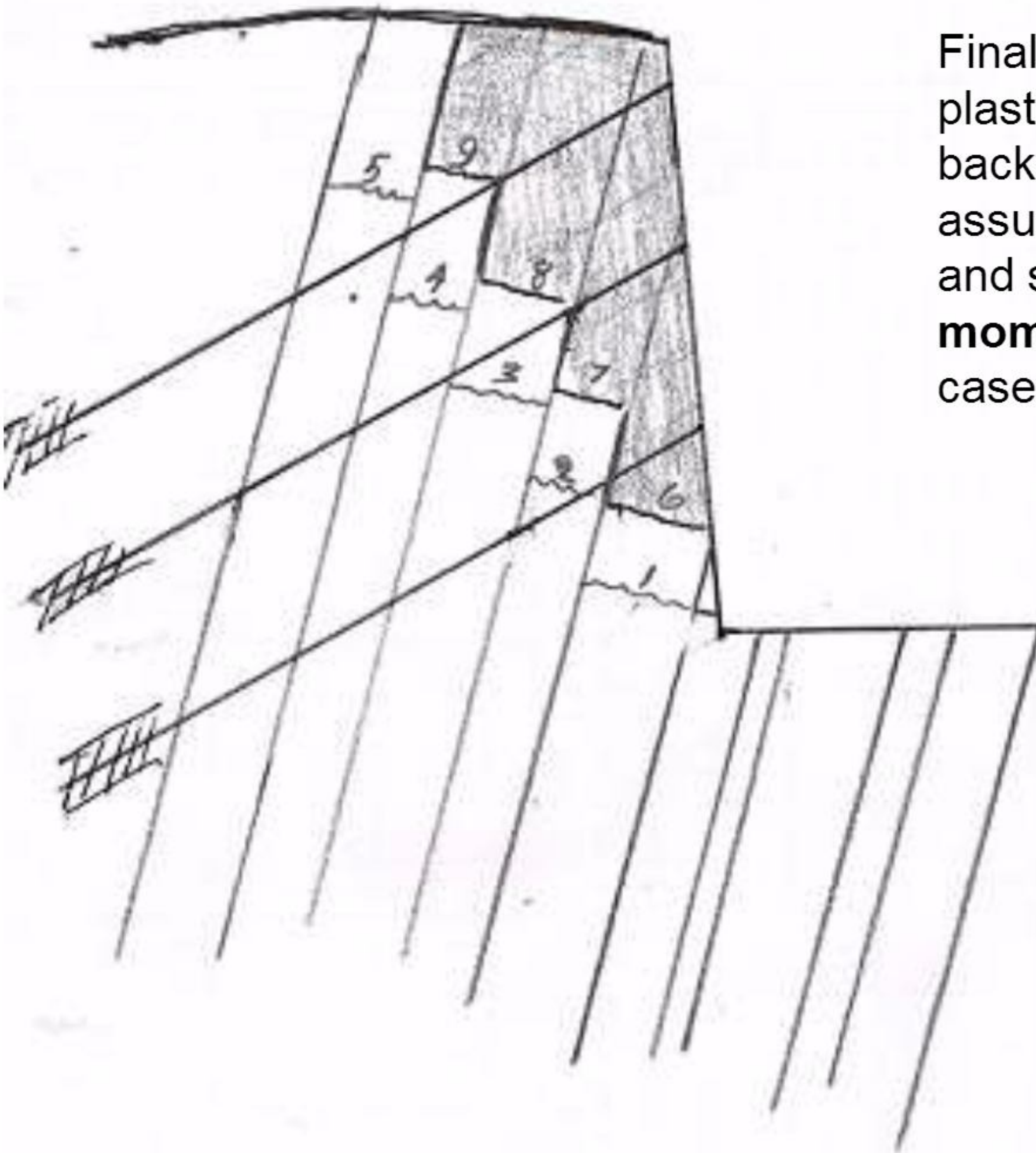
It is hard to realize an **underpinning** approach " for rock slope excavation, but one could be conceived using re-minable, counter-sunk bolts or cables.

The **elastic design for *flexural toppling*** can be achieved using the top-down method of rock excavation, *as long as it can be guaranteed to preserve the joint columns*, **and** *it can be assured that there are no pre-existing cross joints*.

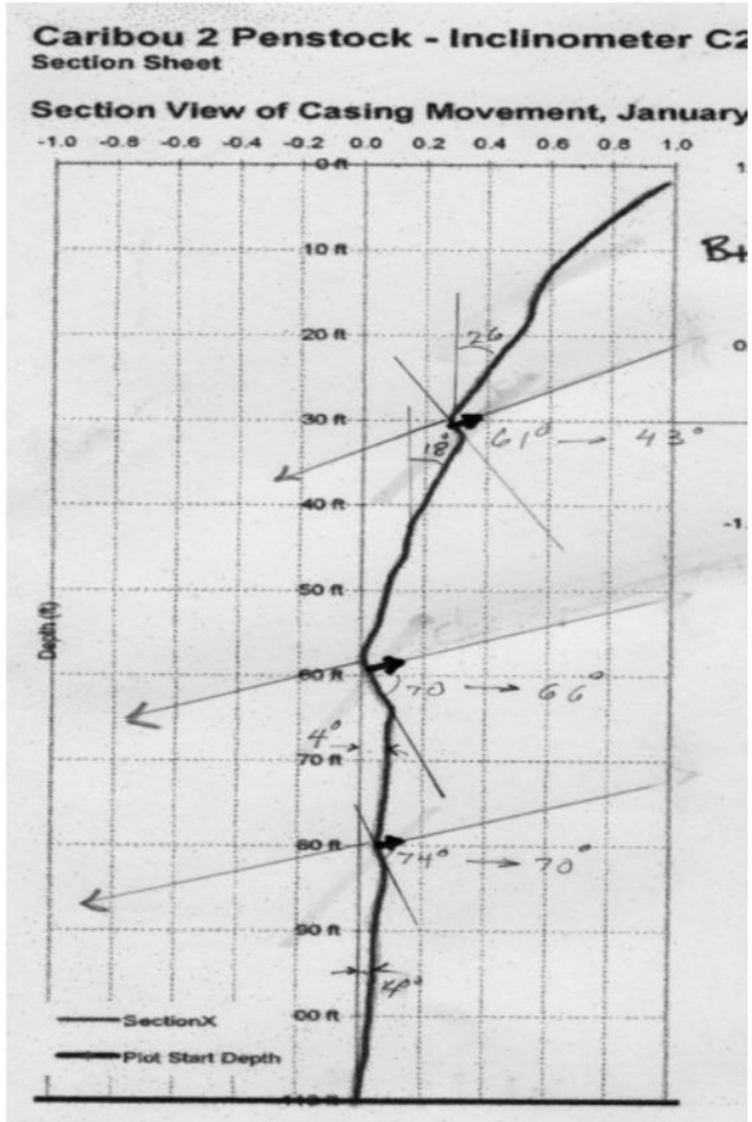
If the cut is completed without staged support installations*, or the existence of cross joints can not be ruled out,* a **plastic design**, corresponding to ***block-toppling*, should be preferred.**

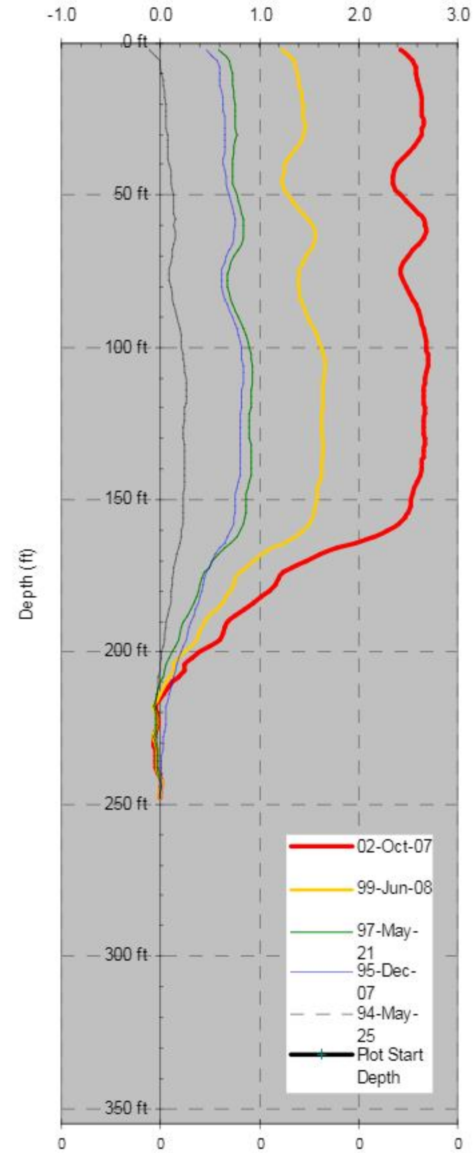


For the plastic design, one or more configurations of block columns can be studied , based on the geologic data. Careful field observations and exploration are warranted.

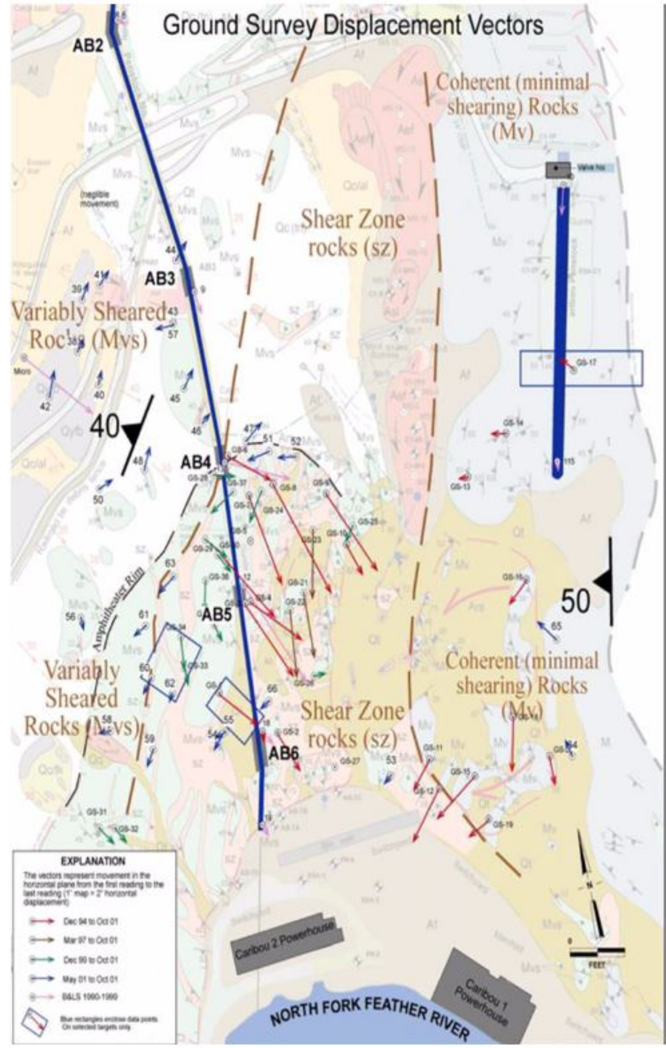


Final configuration of the plastic design. The tie-backs are selected to assure safe anchorage and sufficient restraining **moment** for each load case.





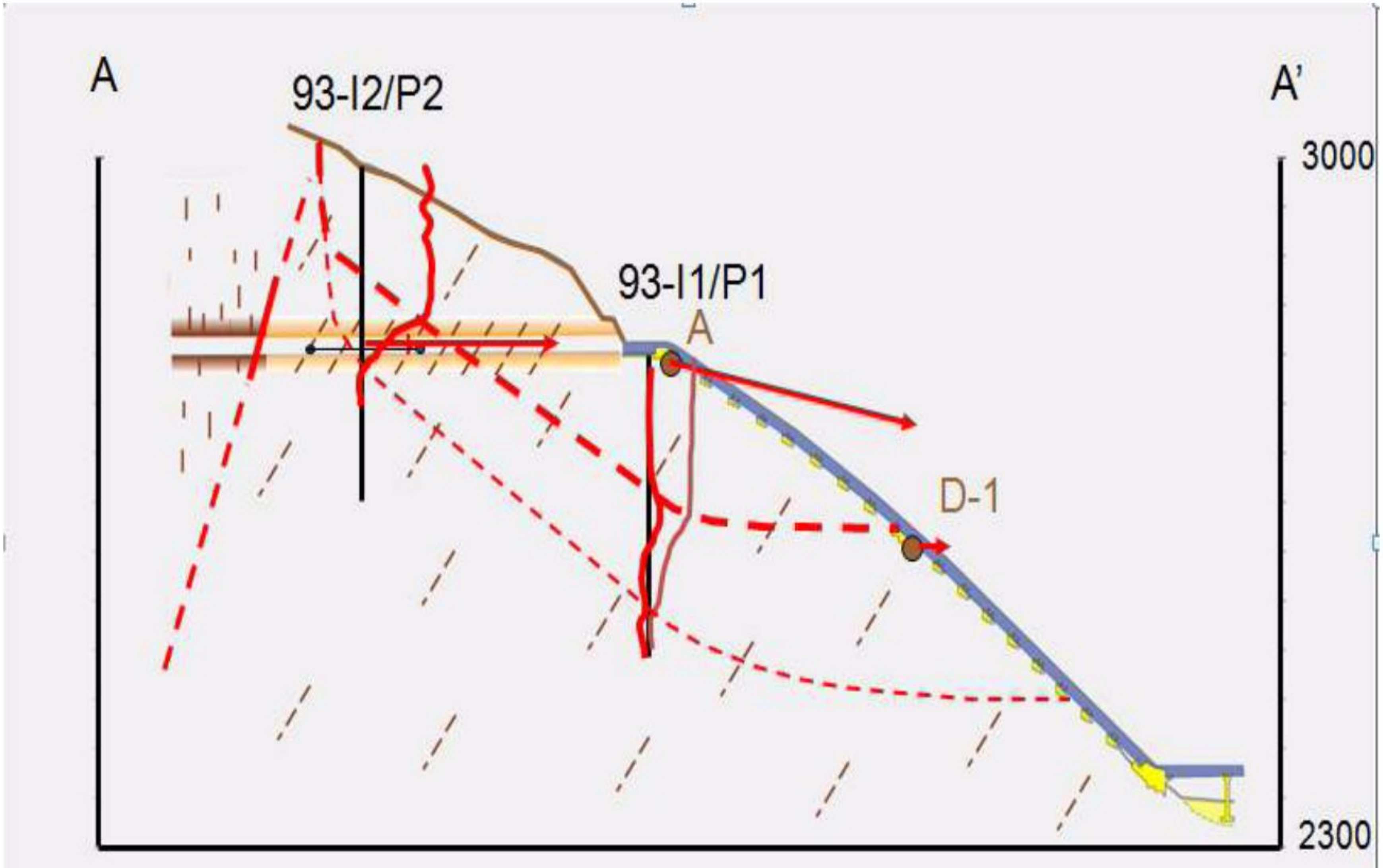
Inclinometer records in boreholes cutting through toppling phyllite formations at PG&E's Caribou (left) and Belden (right) power projects. The wiggles are introduced by flexural slip, and slip on new fractures crossing the toppling layers


Ground Survey Displacement Vectors
AB2
AB3
AB4
AB5
AB6
Coherent (minimal shearing) Rocks (Mv)
Shear Zone rocks (sz)
Variably Sheared Rocks (Mvs)
40
50
Amphitheater Rim
EXPLANATION
Caribou 2 Powerhouse
Caribou 1 Powerhouse
NORTH FORK FEATHER RIVER

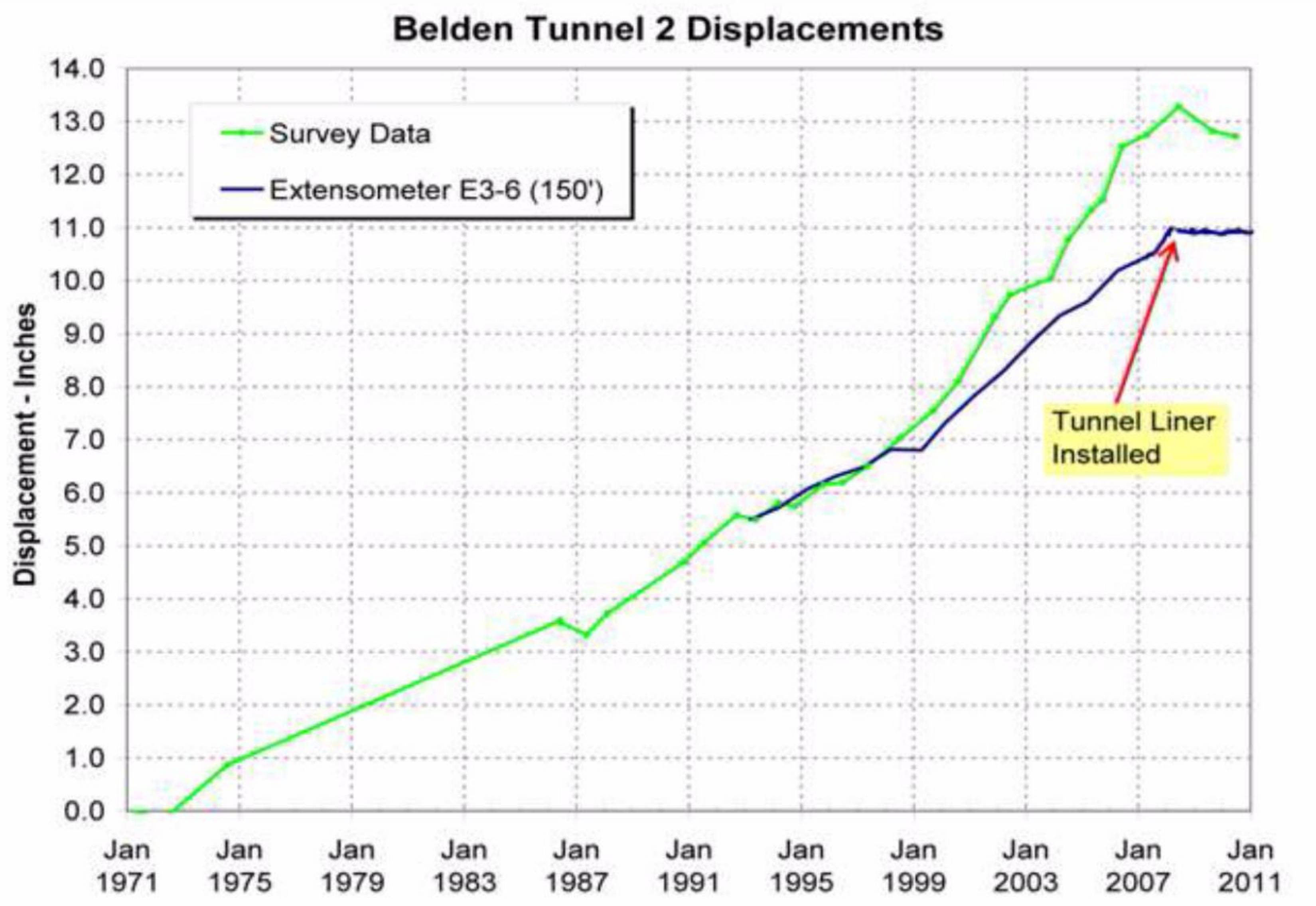

?
02.10.2014


BELDEN TUNNEL 2 PORTAL

Crack in the lining of Belden Tunnel



Belden Tunnel Instrumentation shown in a geologic section


Belden Tunnel 2 Displacements
Survey Data
Extensometer E3-6 (150')
Tunnel Liner Installed
Displacement - Inches
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
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Jan 1971
Jan 1975
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Jan 1983
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Jan 1991
Jan 1995
Jan 1999
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Jan 2007
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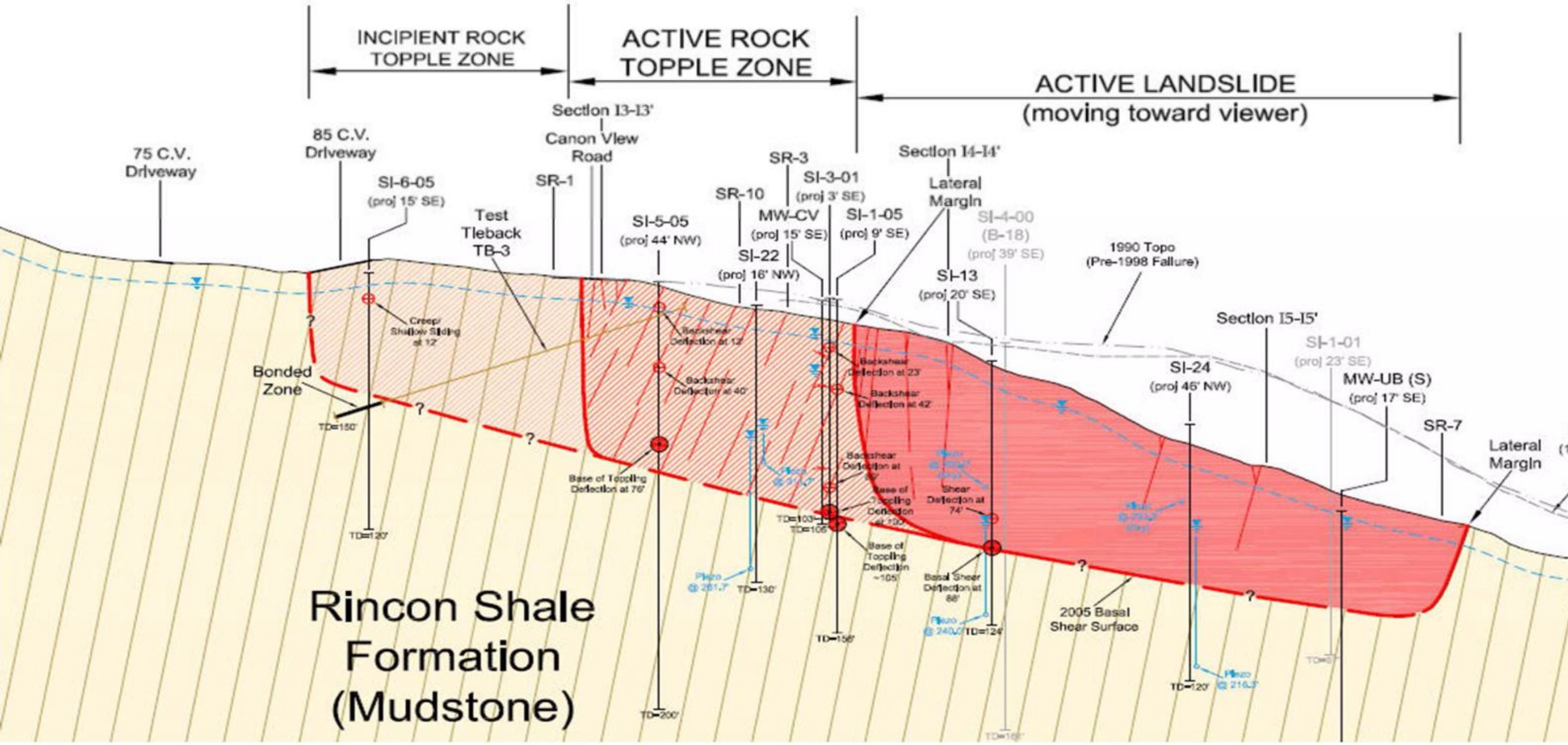
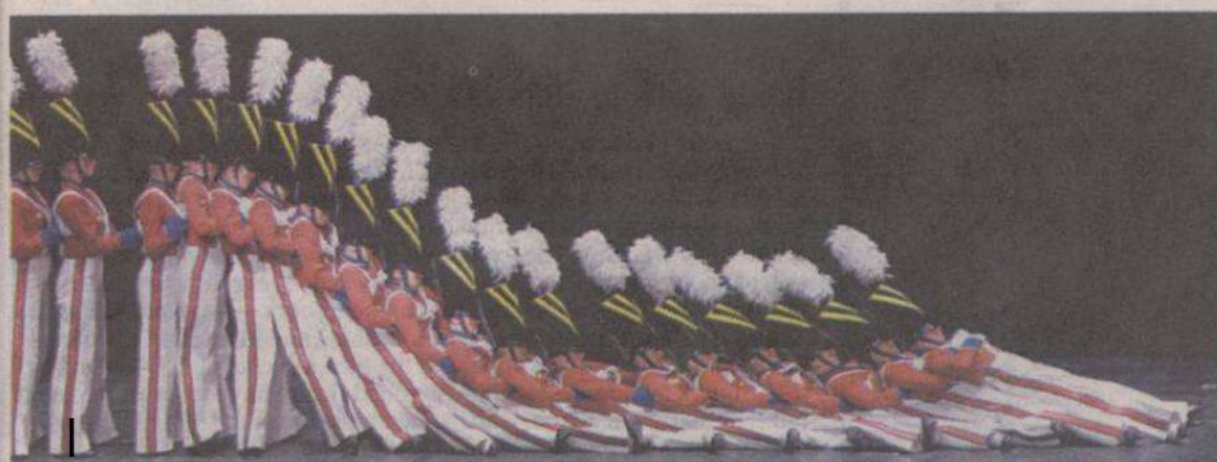


Vertical section of the Santa Barbara landslide looking upslope (Eastward), opposite to the direction of slide motion. Drawing by Cotton-Shires & Associates

Severe damage from development of obsequent scarps by leftward toppling

-- Tied-back walls designed by Cotton- Shires to provide resisting moment to the incipient toppling behind each wall

Toppling



ALI GOLDSTEIN

Slumping

"Toppling - - a fundamental failure mode in rock masses

By Richard E. Goodman, Dr. h.c., Prof. emeritus, Dept. of Civ. Eng., Univ. of California, Berkeley, CA

Toppling of rock blocks, individually, or in rock masses of great volume, is now understood to be an important potential mode of failure for rock slopes. Toppling can also occur in foundations, tunnels and underground chambers and on a small scale in any rocky landscape where frost, creep, or water forces are at play. Toppling failures can develop slowly as an expression of creep, or they can occur suddenly and powerfully.

General recognition of toppling by engineers as an important mode of failure for jointed and fractured rocks developed only in the late 20th century. While geologists much earlier noted and mapped regions of overturned bedding and foliation in sedimentary rocks, schists, and slates of steep mountains - - they tended to attribute their origin to sustained, slow creep of mountain slopes. Engineers seemed to have eyes principally for hazards of rock *sliding* on planar bedding, joints or fault surfaces. Rock engineers and geologists are now attributing to toppling (rather than to recent tectonics) some structural lineaments of vast scale in the flanks of mountains.

This paper describes the features of toppled rock masses in nature, and in base friction models - - distinguishing between *primary* and *secondary* toppling, and *block-toppling* versus *flexural toppling*. Methodology for calculating the potential hazard, and necessary strengthening of rock masses capable of toppling are described in the context of design of excavations. Three instructive California cases are reviewed in brief.

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1. INTRODUCTION – Development of knowledge about toppling failure modes

The design of **slopes in soils** has historically been wedded to mechanisms of *shear failure* under the action of various directly applied forces, *in-situ* stress conditions and a measurable or assumed distribution of water pressures. Historically the principal mode of failure observed in soil slopes involved either *planar sliding* along a shear surface or *backward-rotational sliding* of a semi-continuous mass of earth on a curved surface; sliding masses in both modes typically are cut off at the top by one or more tension cracks. With *rock*, it was well appreciated by civil and mining engineers that the chief mechanisms of failure involved sliding along geologic planes of weakness such as bedding planes, faults, long joints, and pre-existing shears. All geologists were conversant with bedrock-folds and flexures but new folding as an active process was generally overlooked as a hazard in the current landscape. Even Terzaghi, who had sketched toppling failure in a consulting report in the 1950's for a Brazilian quarry, omitted any reference to folding as a mechanism of rock slope failure in "hard, unweathered rock" in his article devoted to that subject near the end of his career (Terzaghi, 1962)

However in the case of **long term creep** in mountain slopes, Terzaghi did recognize intricate flexural deformation controlled by the structure of rock, writing: "... *the laws which determine the deformations are as different as those of hydraulics and of the mechanics of elastic solids. If a system composed of strata with very different elastic properties is acted upon for a long time by shearing stresses which are smaller than the average shearing strength of the system, the most rigid members only will behave like solids, whereas the balance will be deformed like a very viscous liquid*." (Terzaghi, 1950). This point of view may have influenced later observers of *Sackungen*, for example Zischinsky,(1966-1 and 2) who compared his mapped pattern of tightly folded beds, outcropping down the mountain sides, to a hydraulic pattern of creep deformation.

Every designer of a retaining wall or dam knows to consider failure modes both of *sliding (*on its base or in its foundation), and *overturning* ("toppling") about its toe. The stress distribution in a dam or retaining wall reflects both shears and moments about the toe. Similarly, both sliding and overturning are equally applicable to steeply dipping beds or columns of rock, any one of which can act as a retaining wall for its continuous uphill neighbors. Bending and flexural rupture of geological strata and foliations are not only common threads of crustal deformation; they are now recognized as occurring widely at low stresses in valley sides and rock excavations for civil and mining works. Further toppling can occur in the periphery of underground openings and can destroy an underground excavation.

Cruden (1989) noted that the eminent French engineering geologist M. Lugeon as early as 1922 had recognized the potential instability of steeply dipping strata that strike parallel to valley sides. He saw that this situation generates slabs of rock delicately poised, with potential for opening along the bedding and falling under just their own weight. Such layers are readily destabilized by the addition of water forces, the action of vegetation, ice growth, etc. He advised against siting a dam in any valley that runs parallel to the strike of vertical bedding.

Under just their own weight, strata dipping steeply into a hillside have a toppling tendency about the the toe of each block or slab. However, even joints, foliation, and bedding that have the same **direction** of dip, but a steeper **angle** of dip than that of the hillside (referred to as "cataclinal slopes" by Cruden), can topple if subjected to joint-water pressure or other forces in addition to their own weight. Thus, water pressure in open joints between slabs, soil creep over the tops of slabs, ice action and inertia forces can greatly extend the overturning tendency of slabs and columns in the structure of a rock slope, and thereby generate toppling of rock masses of small or great size.

Figure 1 shows the appearance of a toppled slope in a slate quarry near Dnorwik, Wales (G.B.). A pervasive extension crack opened along the slaty cleavage, which dips steeply into the slope to the left, thus releasing resisting moments with rapid overturning of a large rock mass about the toe of the slope. Along the strike of the slaty cleavage, a much larger extension crack can be seen to have formed at the top right - - which prepares the rock for another toppling failure farther to its right to be faced by the unlucky miner who might dare to excavate there. At a much smaller scale, Figure 2 shows forward toppling of slabs of granitic rock near Mont Blanc (Aiguille du Midi), toppling rightward and opening large extension cracks; these may have been pried loose by ice in vertical extension cracks; blasting can cause similar block rotations. Figure 3 shows the Cardinal River Coal Mine, Alberta, Canada, after Cretaceous shales, sandstones and coal beds had rotated from initial dips of 70° to final dips of 30° into the hillside. This gradually accumulating rotation, culminated in two accurately predicted rock falls totaling, 573,000 cubic meters. These latter events, derived from the broken mass of toppled rock, ultimately caused the mine's abandonment Note the progression of long extension cracks visible well up the mountain side. Wyllie and Munn (1978) careful monitoring of rock movement rates permitted safe mining operations to continue for more than a year.

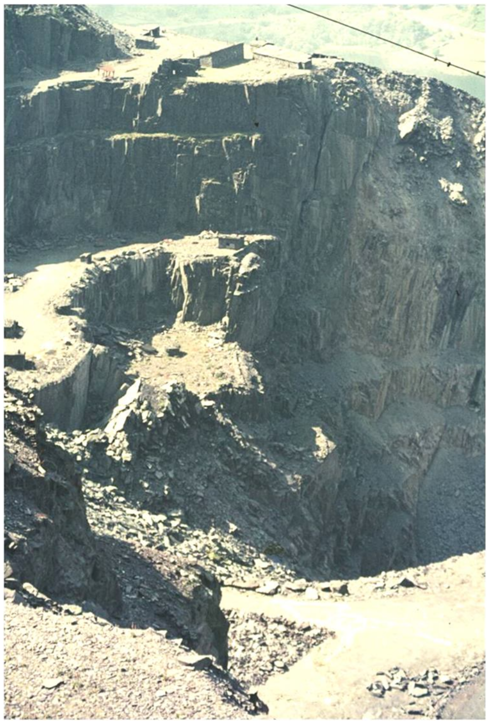


Figure 1 – Toppling in a slate quarry slope, North Wales (Great Britain).

Figure 2.) Toppling and opening of extension cracks in outcrops on Aiguille du Midi, near Mt. Blanc

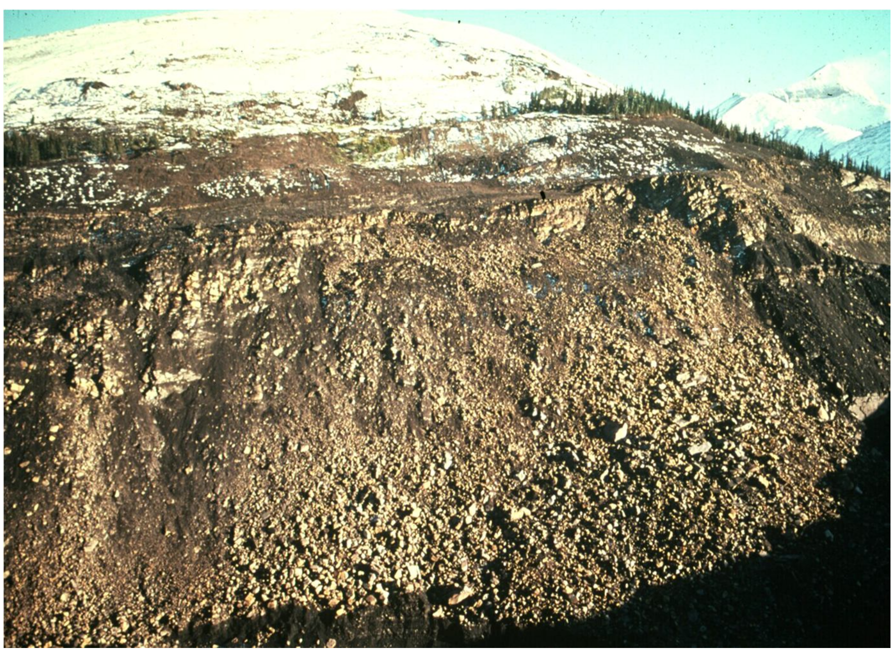


Figure 3 - The failed wall of Cardinal River Coal Mine, Alberta; photo by Duncan Wyllie

2. Morphological and Structural Features of Toppling Failure Modes

An important paper by deFreitas and Watters in 1973, describing toppling failures in Great Britain and various physical model studies around this time, brought the geotechnical community's attention to the nature and relevance of this newly appreciated failure mechanism for jointed rocks . The development of the ideas in this paper were fostered by Watters' Doctoral Dissertation (Watters, 1972) in the Scottish Highlands, and de Freitas' investigative field work in the coal-measures' strata along the coast of North Devon, England. Also significant was a much cited Masters'

Thesis by John Ashby (Ashby, 1971) who studied two dimensional physical models to describe the modes of failure of a rock slope cut in a block-jointed rock mass. This time period followed the provocative and tragic failure of the abutment slope of Vajont Dam,Italy. Professor Leopold Müller's paper "New considerations on the Vajont Slide" (Müller, 1968), included one figure suggesting a local toppling event contiguous to the slide margin. Subsequently, Müller's student Heinz von Hoffman initiated two-dimensional kinematic model studies in jointed rock masses (Hoffmann, 1972 and 1973). Both Ashby's and Hoffman's model studies demonstrated that toppling can occur on surfaces dipping at less than the friction angle of the joints, following an initial sliding motion in the toe of the slope that creates the kinematically necessary space for the slide to move into. The field investigations by deFreitas and Watter's demonstrated that toppling mechanisms do indeed exist as naturally occurring events and require no unusual geologic conditions for their development - - only the transfer of excess shear force from the limiting equilibrium condition high in the slope into developed shear stresses and sliding displacements in the toe of the slope that triggers collapse of potentially toppling columns above. This is shown in Figure 4, a section through deFreitas and Watters' Glen Pean topple, in schistose metamorphics, with a volume of the order of 30 million cubic meters.

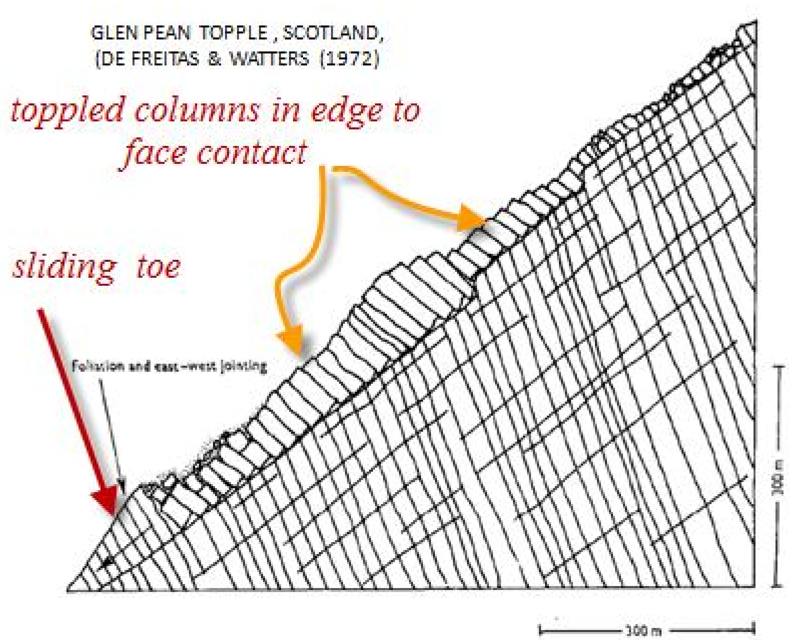


Figure 4 – A diagrammatic section through the Glen Pean Topple - -adapted from deFreitas and Watters (1973)

In coastal cliffs, as in deFreitas and Watters' toppling slopes of North Devon, quarrying of outcrops by ocean waves can effect the same sort of destabilization of the contiguous uphill mass as toe sliding in Glen Pean topple. As seen in Figure 5, wave attack at the base of a sea-cliff tends to loosen and remove key blocks forming the toe of the slope. Subsequently neighboring blocks, resting on a base plane inclined at angle α with horizontal, will tend to topple if the ratio of block width (b) to height (h) (governed by the individual joint spacings) is less than tan α. Thus the kinematic necessity for toe sliding to precede gross toppling failure may be absent in coastal rock cliffs, contributing to the general hazard for homes with a view along an ocean or lake shore.

Toppling will occur above a surface inclined at angle α when the ratio b/h <tan α. Marine erosion removes Block A, allowing B to topple; Blocks C and D follow but E remains, as it has a wider base.

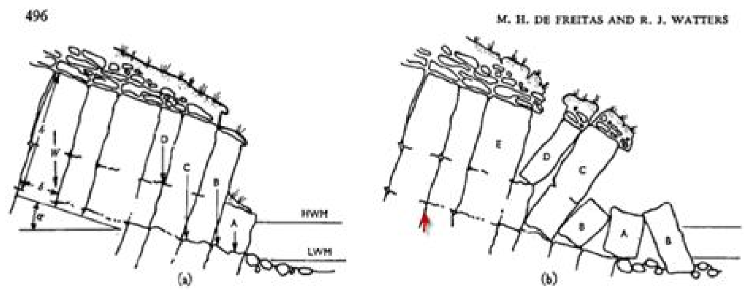


Figure 5 - Condition for overturning under self weight - - from deFreitas and Watters (1973)

A main characteristic of a toppling failure is the development of *obsequent scarps* - - normal-fault type extension scarps (also called *anti-slope scarps,* and *upslope facing scarps)* . They occur at the top and behind the toppling mass with strikes sub-parallel to the topple-crest and dipping away from the steep, free-surface. Approaching the top of the scarp from behind, a succession of these scarplets typically greets one as a set of stairs with gently inclined tread, as can be seen in Figure 6. They characteristically display normal fault offsets - - with the toppling layers moving upward and outward relative to the rock behind the topple. [1]

Figure 6 - *Obsequent*, or *antislope* scarps (uphill facing normal faults) that typically form by rotational extension behind a toppling face.

In the 1960's, I observed a small topple in creeping soils of the Berkeley Hills, and visited a large toppling slope in phyllites in an overly steep highway cut of the Oroville Dam project, California. Subsequently in the 1970's, I was fortunate to spend a sabbatical leave and several follow up visits with Prof. Evert Hoek's group at Imperial College, London, where I became acquainted with the outstanding research being conducted under his direction in rock mechanics, with some collaboration from the soil mechanics and engineering geology groups. I read about and visited toppling failures on the coastal bluffs of N. Devon and in open pit mines of Wales and Cornwall These experiences, and extensive discussions with faculty members and students, led to a research paper with Prof. Hoek's faculty colleague John Wade Bray on classification and stability analysis of potentially toppling slopes (Goodman and Bray, 1976).

In this paper, we recognized a *potential topple* as a blocky rock mass appearing to have the likelihood of transferring a significant thrust force into the toe of a slope by virtue of each block's tendency to lean against its downslope neighbor. If the cumulative force transferred in this fashion to the toe of the slope were sufficient to dislodge the toe by sliding or overturning, it would bring down rock columns from above. This would happen automatically if the structure of

[1] The author first saw this behavior in the failure of a large limestone block in the temporary excavation of One Main Place, Dallas and attempted a finite element analysis with Barry Voight, then of Dames & Moore.

the rock mass divided it into steeply dipping layers, chopped by crossing joints or fractures to create columns of stacked blocks, as in Figure 7 a . We named the toppling failure of such a mass ***Block Toppling***.

Additionally, a layered, foliated, or bedded rock that lacked cross-cutting joints or fractures, could develop crossing fractures by bending (*flexure*) and cracking as in Figure 7b; we named this behavioral mode ***Flexural Toppling***. Figure 7c shows a third behavioral class - - ***Block-flexural Toppling*** ; in the latter case, multi-block columns fail by pseudo-flexure, the apparent bending consisting of numerous discrete adjustments on the cross joints that effect differential rotations of individual blocks rather, than by continuous flexure of the rock itself.

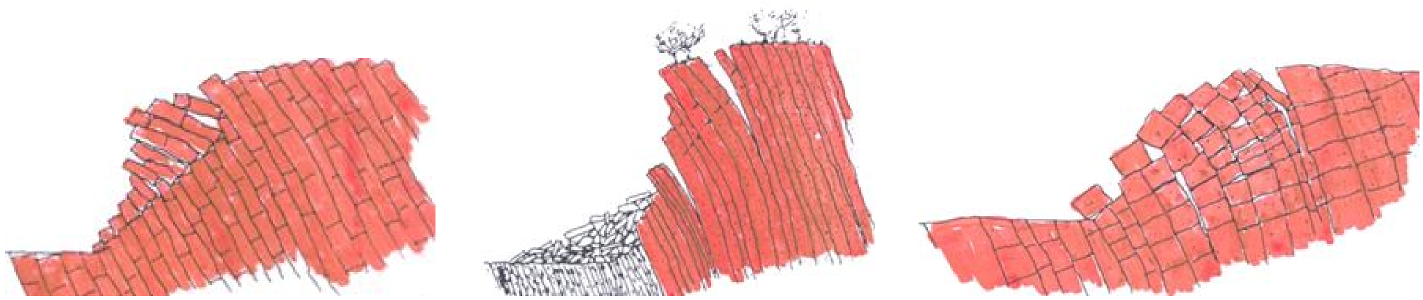


Figure 7 - Toppling modes: left, *Block Toppling*; Center, *Flexural toppling*; Right, *Block-Flexural Toppling*

Block toppling failures can be sudden and violent while flexural toppling may be even more so, unless initial block movements self-stabilize as the deformations reduce the potential for continued motion. On the other hand, all toppling masses tend to develop new fractures and to rotate previously quiet joints and fractures into steeper orientations, both of which can trigger rock falls, rock avalanches, and slides. Examples include: the final rock failures at Cardinal River Coal Mine, described earlier; the Mystery Creek rock avalanche, B.C, discussed by Nichol, Hungr, and Evans (2002); five cases described by Chigira and Kiho (1994) in Honshu, Japan, two of which generated avalanches during earthquakes; and an avalanche in toppled limestones described by Hu and Cruden (1992).

Some rock structures suggest the potential of toppling immediately on inspection of the cross section. Others, which we denote as ***secondary topples***, become potential toppling structures only after movement of a contiguous mass triggers a potentially dangerous load or new space. Figure 8 shows four types of *secondary topples.* ***Slide Head Toppling*** is capable of occurring behind a landslide, whose movement creates a void into which slabs of the upper, formerly contiguous material may topple. Similarly, ***Tension crack toppling*** can occur when a landslide or slump pulls away from the scarp, allowing overturning of the slabs behind the scarp between parallel tension cracks. ***Slide base toppling*** can happen when a slide or slump drags transversely across a joint or free surface that intersects a number of vertical or inclined joints, causing toppling of the slide's foundation rock . Finally, ***slide toe toppling*** can occur when a slide thrusts against the boundary of a structurally different rock mass with steeply dipping bedding, cleavage or jointing.

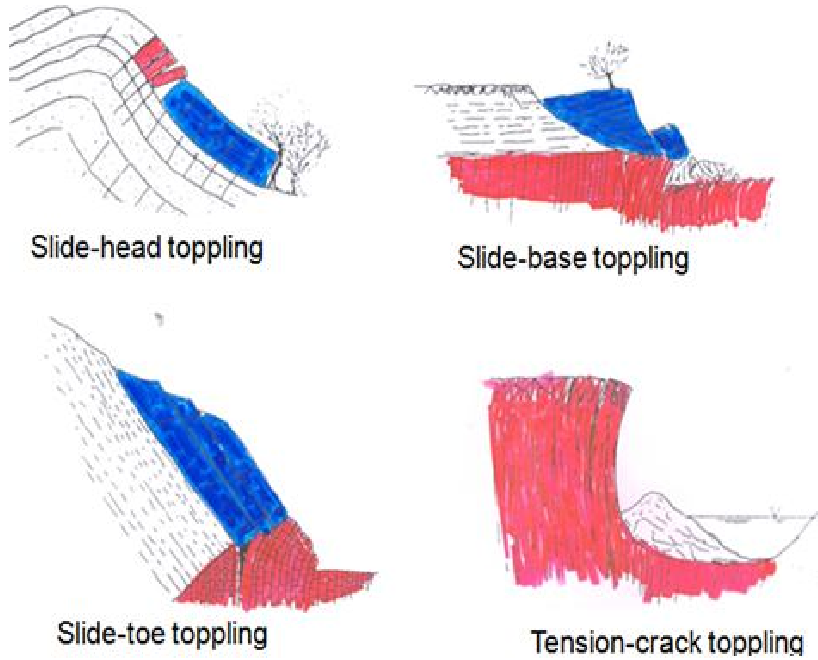


Figure 8 - Secondary toppling modes, from (Goodman and Bray, 1976)

Although named *"secondary topples", such events are not necessarily secondary in importance*. For example, an impressive working group of scientists and engineers, conducting an intensive study of the kinematics of the dangerous Randa Instability in Switzerland, (V. Gischi et al, 2011), has concluded that the current rock fall hazard there is due largely to secondary toppling from the main scarp of the 30 million cubic meter rock fall of 1991 (Figure 9). It was reported that there are 5 to 6 million cubic meters of unstable rock exposed in the inaccessible, 850 meter high scarp .

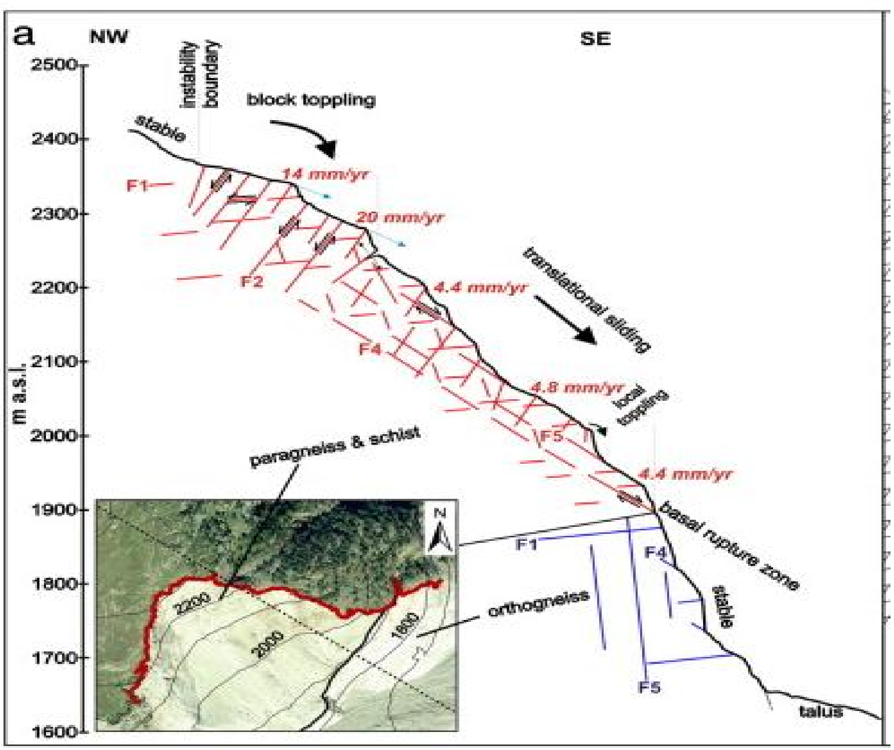


Figure 9 - A vertical section along the high scarp of the 1991 Randa Instability, Switzerland, and inset plan. Reproduced from (Gischig, Amann, Moore, Loew, Eisenbeiss, and Stempfhuber 2011)

Another form of secondary toppling - - **sliding-induced toppling** - - can initiate from deformation of the rock mass caused by initial sliding. Figure 10 is a base friction model of toppling that is generated by incipient sliding along non-circularly folded bedding. The base friction model, (Bray and Goodman (1981)) allows easy and quick repetition of two dimensional experiments facilitating study of relative influences of surface geometry, joint spacing, roughness, orientation, and strength characteristics of several sets of joints. The results can be strictly correct only for static equilibrium, such that no block acquires significant momentum. That limitation proves generally non-consequential because most of the blocks are within the interior of the simulated rock mass and cannot acquire momentum until large deformation has already been achieved. Thus results observed during an experiment are acceptable, except for blocks that may be "falling" into the free space.

In the example of Figure 10, incipient shear creep along the layers causes toppling of the toe and complete destruction of the entire slope. In Figure 10a, we can see that sliding begins to develop interbed gaps which set up flexural cracking of certain beds. As this proceeds, the potential sliding mass begins to over-ride the toe, causing foundation crushing beneath the toe of the slope (Figure 10b). The consequent rotation of the toe opens imbricate flexural cracks delimiting a major toppling mass. Moment is now delivered from the rock mass behind and above the topple (Figure 10c). This culminates in a complete destruction of the rock slope (seen in subsequent frames not copied here).

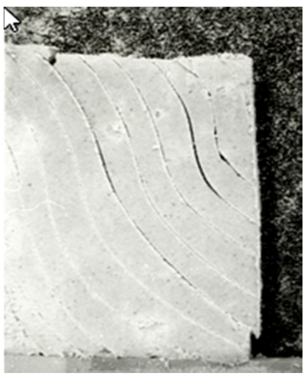


Figure 10 a

Figure 10 b

Figure 10 c

Figure 10 d

Conventional block toppling , as described earlier, usually occurs when *toppling in the head of a slope is allowed by sliding in the toe*. **Kink-band slumping** (Goodman and Kieffer, 1998), sketched in Figure 11, is an obverse failure mode in which *sliding on bedding in the head of a dip-slope is enabled by formation of a kink-band and overturning of the same strata in the toe*. Its analysis is discussed in Kieffer's doctoral dissertation (Kieffer, 1998). As viewed from below, the strata would seem to suggest that a toppling failure had occurred on strata that do not dip into the slope. [2]

[2] Cruden (1989) referred to this situation as toppling on *an underdip, cataclinal slope.*

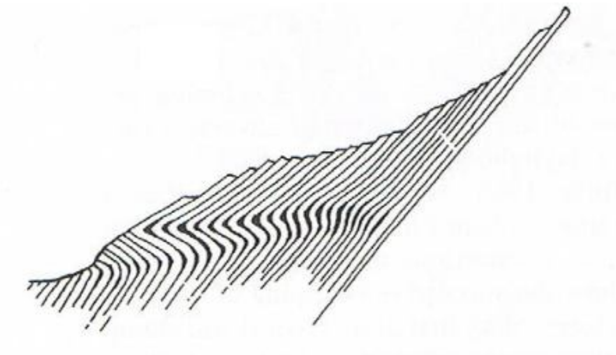


Figure 11 – Kink-Band Slumping

3. Toppling-induced Buckling:

Buckling is a failure mode applicable to long, slender, steeply inclined columns, often of weak rock. It is a known hazard in open pit mining of steeply inclined coal beds. Removal of coal prejudices the stability of the weak strata that typically lie behind (underlie) the coal. Buckling can also be initiated by toppling, as individual rock columns separate from the rock mass. A column that is cross jointed, or has accumulated flexural cracks that cut almost entirely across the column (as in Figure 10), can be said to be formed of *stacked sub-blocks*. As toppling rotations develop, the upper sub-blocks of any column may generate insupportable moment at one or more intra-column hinges as shown in Figures 12 and 13.

Figure 12(a)(left) - Initial cut-slope, in a block-jointed rock mass with one set dipping steeply into the hillside.
12(b)(right) - Buckling initiated by toppling of this slope.

The three layers of toppled slopes: It has been noted, in several engineering works constructed on toppled slopes, that geophysical measurements tend to be interpretable with a site model comprised of three offset layers with differing properties. Field observations, and analytical and physical modeling support this picture. Toppling with induced buckling provides an ideal three layer case as shown in figure 13. The upper portion (1) becomes a dead weight riding on the toppling mass (2), with a relatively undisturbed zone (3) at the base. Notice also, both in the model and the actual rock topple shown in Figure 13, rotated beds of layer 2 stand in edge-to-face contact with the rock surface of layer 3 - - the base of toppling.

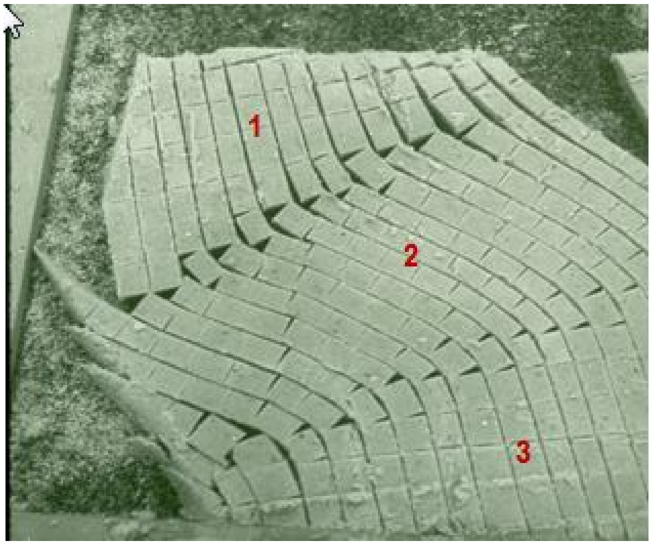
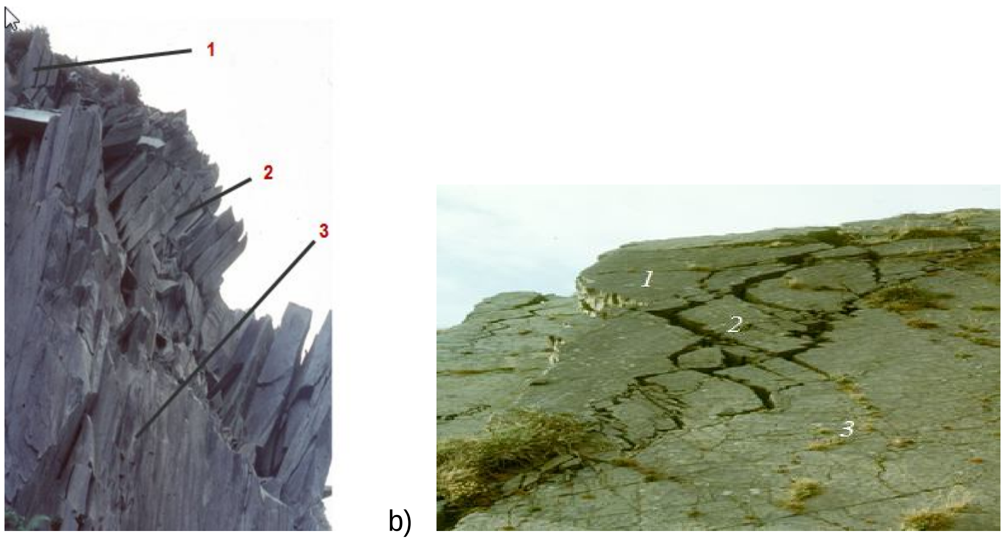


Figure 13 – Base friction model slope with orthogonal joints, subdivided into three layers by toppling-induced buckling .



a) b)

Figure 14 –Three offset layers of toppling exhibited in rock exposures: a) an exposed section through a topple in slate showing three layers (note also the edge/plane contacts at the base of zone 2); inclined beds undergoing toppling-like movement of the upper layer. In this mode, properly termed "block torsion", added frictional restraint - from the inclined, under-side of the topple - preserved the open toppling joints from immediate failure.

3. Toppling as a mechanism in large scale deformation of mountain ranges

It is important to distinguish the *mechanistic* discussion of toppling here from any *genetic* discussion of gross mountain deformation. A number of publications by geologists and engineers, including those by Stiny (1941); Zischinsky (1966-1&2); Radbruch, Varnes and Savage (1976); Varnes (1987); Hutchinson (1988); Poisel (1998); and Reitner, Lang, and Van Heusen (1993), discuss the origin and implications of Recent Age mappable structures in deformed, faulted, and/or eroded rock masses of the Alps and other great mountain ranges. Geomorphic and structural processes responsible for these features have been tagged by various names, including *Talzuschub* (Valley thrusting), *Sackungen* (Saggings), *Lateral spreading*, and *Mountain splitting*. The origin of these structures have been attributed to various mechanisms including *deep-seated creep rupture*, *large scale toppling*, *foundation failure where stiffer formations overlie softer ones*, *destabilization of slopes by rapid erosion following deglaciation*, and *Recent tectonism*.

Progress in the fields of geomorphology, structural geology, and engineering geology is fostered by informed conjecture about the origin of large scale post-genetic mappable structures in mountains and their potential societal implications. Of immediate interest to this paper is the fact that a number of workers are now describing and mapping what they interpret as large scale toppling phenomena in mountain slopes. For example Hippolyte et al (2006) interpreted mappable normal fault scarps of Pleistocene and Recent age in the French Alps as obsequent scarps of gravitational toppling, arguing against interpreting these features as evidence of active tectonism, with all its potentially expensive implications.[3] Figure 10 shows their interpretation of the gradual development of progressive bulging of the lower slopes, normal faults as antislope scarps, and tension splitting of the ridge top in response to the toppling on both flanks. The overall pattern might well be called *Mountain Sagging* while the structural failure mechanism can be labeled as *toppling.*

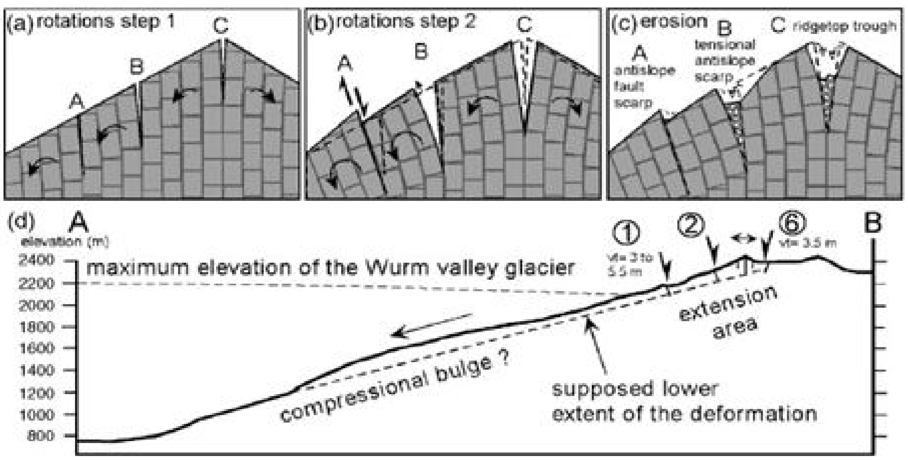


Figure 15 – Toppling as a mechanism for Sackungen; reproduced from Hippolyte et al (2006)

4. Analysis of Block Toppling Failure

The first published analysis of block toppling, to the author's knowledge, was the spread-sheet, limit-equilibrium approach published jointly with Imperial College faculty member John Bray (Goodman and Bray (1976)). It was developed specifically for the kinematics of a series of columnar blocks resting upon an upwardly stepping base as shown in Figure 16. In this configuration, any column may tend to slide, or to rotate about the edge of a step, as shown in the figure. We studied also the case of toppling on an inclined, *plane* base, but this proved to be far more difficult than the stepped base because *upslope rotation* of a block must in general be allowed and its testing necessitated complicated cycles of recursive logic.

The Goodman and Bray analysis computes downward from the highest block that tends to rotate or slide under the prevailing forces (see Figure 5). Using equilibrium equations for toppling and for sliding of this block, the reaction forces from the lower neighbor block at limiting equilibrium in both overturning and sliding modes are calculated and the greater of these two candidates is accepted. Its *vector opposite* acts downslope as a driving force on the second highest block. Proceeding in this manner for the second block, and subsequently each block down the slope, determines the complete succession of interblock forces for the entire system of blocks.

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[3] . Discovery of valley side scarps above the site of the high Mica Dam in British Columbia, during the 1960's, created some consternation for the design engineers. Toppling as a mechanism of slope failure was not common engineering knowledge in those years. It was held out that these features might be hitherto unrecognized tectonic fault scarps, and accordingly that the design might have to be altered to accommodate a stronger ground motion. Based on detailed, but somewhat indecisive geologic studies, it was eventually concluded that the features were of an unknown origin *other than* active faulting.

The value of the reaction force P_o thus calculated for stability of the *lowest* block, measures the stability of the whole group of blocks: if it is negative, the system is in equilibrium without additional support; if zero, the system is at limiting equilibrium; if positive, the block system is unstable and will fail if the bottom (toe) block is not supported. According to whether the toe block is sliding or toppling (meaning whether the minimum required reaction force for the toe block is derived from the equilibrium equation for sliding or for toppling), safety can be provided by acquiring shear or moment resistance with an anchored wall or appropriately directed tie-back. Alternatively, safety can be sought by binding pairs or triplets of blocks together with rock reinforcement to effect an increase in the apparent thickness of columns; however this may be hard to realize in practice and may not prove to be cost-effective in any particular case..

a)

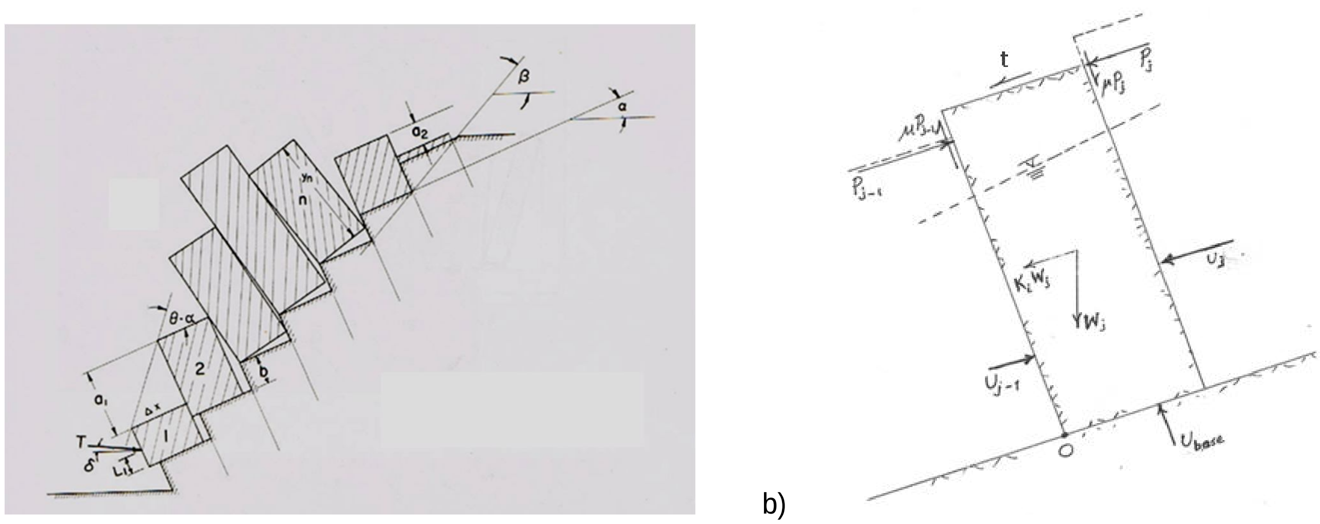


Figure 16 : a) Model for analysis of toppling on a stepped base, from Goodman and Bray (1976). b) Forces on block j with respect to rotation about 0: ***P_j*** *= normal force from upslope neighboring block (j+1);* ***P_{j-1}*** *= normal force from downslope neighboring block;* ⍰ *= block side-friction factor;* W_j *= weight of block j;* ***K_i*** *= acceleration coefficient of an earthquake in the most critical direction (// to base);* ***t*** *= creep force of overlying soil; and* ***U_j, U_{base},*** and **U**$_{j-1}$ *are resultant water pressures on the upper side, base, and lower side of the block respectively.*

It is somewhat misleading to report the factor of safety from this analysis in the usual terms, i.e. as the ratio of the tangent of the available friction angle to the tangent of the mobilized ("required") friction angle. This is because the block-toppling stability often depends more on resistance to overturning of the upper blocks than to enhancement of inter-block sliding along the block sides, or friction acting along the base of the potentially sliding toe-blocks. Alternatively, in a case where tie-backs are used, the load factor of safety can be expressed in terms of the ratio of applied moment to be supplied by the tie-backs to the minimum tie-back moment required just to attain limiting equilibrium.

The calculation procedure using Excel spread sheets can be somewhat tedious , and is subject to input/output errors in operation; but it proves practical and general, and can easily be customized for particular conditions such as: earth pressure reaction against a toe wall; particular drainage conditions within the slope; variable joint strengths; variable material properties; internal supports installed between blocks; and particular block shape changes within the system of blocks.

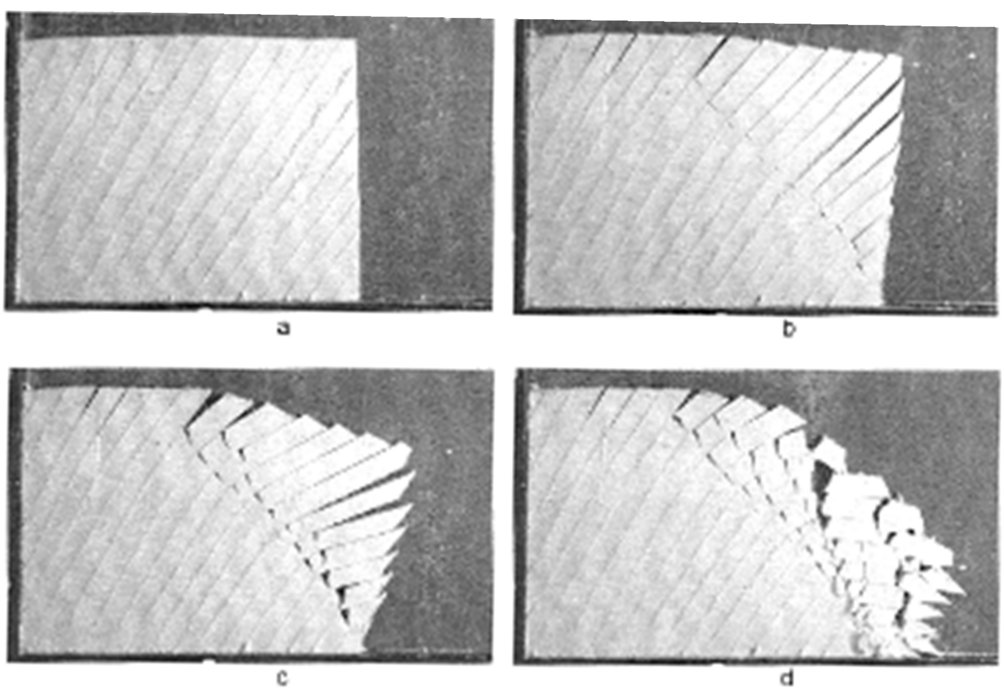
The spread sheet approach was quickly adopted in practice for analysis of block toppling cases and as a starting point for additional developments. This energy provoked significant broadening of analytical capabilities. Ke, Thapa, and Goodman (1994) used the spread sheet results as a check on numerical modeling of block toppling with DDA (with the strength reduction system for determining the mobilized friction angle) and also introduced a soil creep force (**t** in Figure

16a) in their analyses. Bobet (1999) developed an efficient closed-form solution for a particular, symmetric geometry, which may be less cumbersome than the spread sheet approach when the number of blocks becomes large. Zanbak (1983) worked the spread sheet to provide solution charts. Sagaseta et al (2001) solved ordinary differential equations for toppling, cleverly derived by shrinking the individual blocks to infinitesimal thickness; they applied this method for design cases in Spain. Scavia, Barla & Bernardo (1990), and Tatone and Grasselli (2010) developed probabilistic approaches for processing toppling analyses; the latter produced (and offered) the program ROCKTOPPLE for their Monte Carlo analysis. Liu, Jaksa & Meyers (2009) also improved spread sheet computational efficiency. It should be noted that, unlike slope stability calculations based on soil shear strength used in soil mechanics, there is not in general an optimum failure path through the slope for systems of toppling blocks. In other words, engineering geological investigations of the rock structure and strength cannot be by-passed or short-circuited if a credible analytical result is wanted.

4**. Analysis of Flexural Toppling Failure**

Rock columns that lack cross-jointing can fail by bending and flexural cracking, as shown in Figure 17 for a simple vertical slope with regularly spaced joints dipping into the hill. When the cracks have extended to perhaps half the width of each column, the rock mass behavior passes from one of plastic rotation into a mode which may or may not resemble that of an initially cross-jointed rock mass.

Aydan and Kawamoto (1992) published the first limit equilibrium analysis for flexural cracking - an epic paper in applied rock mechanics.[4] The slope was analyzed as a described stack of simple inclined columns, each simultaneously subjected to axial (P/A) and bending (Mc/I) stresses. Prior base- friction model tests by Kawamoto had revealed that columns tended to break off at the base of the largest removable triangular region that passes through the toe of the slope, subsequently failing the entire group of columns . (Their method assumed that group behavior prevails over individual column failure and therefore it is not appropriate for analysis of failure by successive peeling off and buckling of successive face columns as in Figures 12, 13, and 17.)



[4] Aydan and Kawamoto published an earlier paper describing the flexural toppling analysis in 1982, but in the Japanese language only. Unfortunately this paper appears not to have been translated into English.

Figure17 Base friction model showing development of flexural toppling, from Goricki (1999)

From an analytical basis, the authors concluded that "the maximum resistance to be offered by any single column is equal to the load that causes crack initiation at the outer fiber." Starting at the highest inclined column, they calculated the total transverse support force that must be provided to the block from its adjacent lower neighbor in order to preserve equilibrium with respect to the column tensile strength (expressed by the modulus of rupture of the rock). This transverse external force is the integral of a distribution of moment-providing side-forces from the block below arising from gravitational and seismic normal stresses; it was placed at a single point at a distance ηh_i above the base of the column, along the lower side, of length h_i (Figure 18). They also assumed that the flexural crack would propagate in a direction normal to the column sides - - a conservative assumption. Note the non-orthogonal inclination of flexural cracks with the sides of block columns in Figures 10 and 17 of this paper. Centrifuge tests by Adikhary, Dyskin, Jewell and Stewart (1997) showed the actual crack angles in their tests to be 12° to 20° above the normal .

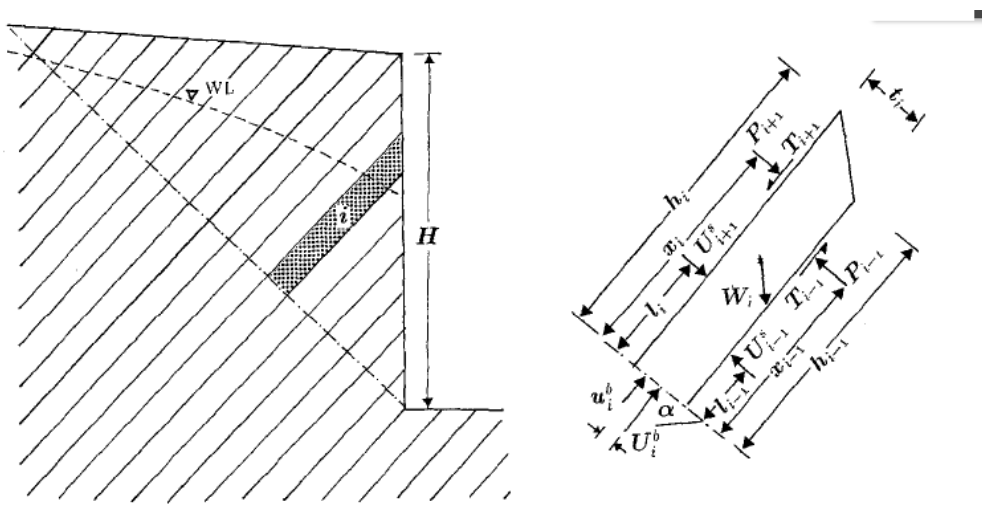


Figure 18 – Kawamoto & Aldan's freebody diagram for limit equilibrium analysis of a flexural-toppling-slope.

Similar to the block toppling analysis discussed previously, the limit equilibrium analysis proceeds block by block through the entire set of columns. The moment-resisting transverse, lower-side reaction forces are calculated for limiting equilibrium in each column starting with the highest. These accumulate to yield the reaction force P_o required to support the lowest inclined column and therefore to maintain limiting equilibrium in flexure throughout the entire slope. As previously discussed for analysis of block toppling, limiting equilibrium in flexural toppling is satisfied exactly, without external support, when $P_o = 0$. The factor of safety is greater than unity if P_o is negative, while the slope requires strengthening if P_o is positive. However, the conditions approaching failure in block toppling as opposed to flexural toppling are quite distinct; block toppling initiates at the toe and spreads throughout the slope, whereas flexural toppling, according to the assumptions of the analysis, is universal and instantaneous throughout the failing mass. Simultaneous flexural toppling of a group of rock columns is therefore likely to be the more sudden and violent.

Aydan and Kawamoto's 1992 paper discusses not only limiting equilibrium analysis for flexural toppling of steeply inclined rock layers in a *slope*, but also as it may occur in the walls and roof of an *underground opening* excavated through inclined strata, or joints. If the variable state of stress around an underground opening is ignored, then the free body diagram of potentially toppling strata in an underground wall could be assumed to be the same as for a similar rock wall at the surface. The free body diagrams for both a tunnel wall and its sloping roof are shown in Figure 19. Both can be analyzed by the limit equilibrium procedure outlined above. The simplicity and directness of a limit equilibrium analysis for toppling in underground openings is an important application that may not even yet be appreciated by the tunnel engineering community.

For both slopes and underground openings, a series of plots by Aydan and Kawamoto show the influence of the principal variables: the dip, thickness, and friction angle of the rock layers; the unit weight, and tensile strength of the rock; and geometric parameters of regular rock bolt supports installed to stabilize the layers. Subsequently, Adikhary, Dyskin, Jewell and Stewart (1997), in discussing back-calculations from their elegant centrifuge model study of flexural toppling failures, presented families of curves giving the critical slope height as a function of the joint dip, and joint friction angle for different slope angles and rock slab thicknesses. Figure 20 is one of 7 such "design charts", produced in 5° increments from ϕ = 10° to 40°. They generalized the results obtained from the charts by means of the following basic *dimensionless* equation connecting the variables : $\mathbf{H_{cr} = [\gamma\ n\ H^2] / [(\sigma_t / F)\ b]}$, in which: $\mathbf{H_{cr}}$ = a dimensionless critical slope number; γ = unit weight of the rock; $\boldsymbol{\sigma_t}$ is the modulus of rupture (tensile strength in flexure) of the rock; **H** is the slope height; **F** is the load factor of safety; **b** is the column thickness (= spacing of column-defining joints or beds); and **n** is a weighting factor to facilitate plotting an iterative solution with increasing unit weight.

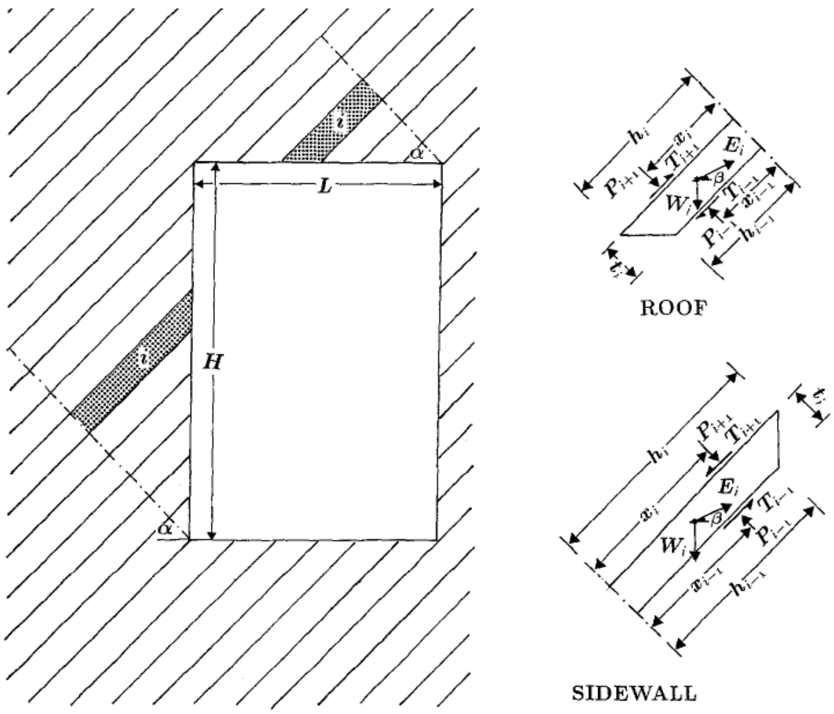


Figure 19 – Aydan and Kawamoto's free body diagrams for the shaded element of a potential flexural toppling tunnel wall, and a potentially toppling tunnel roof.

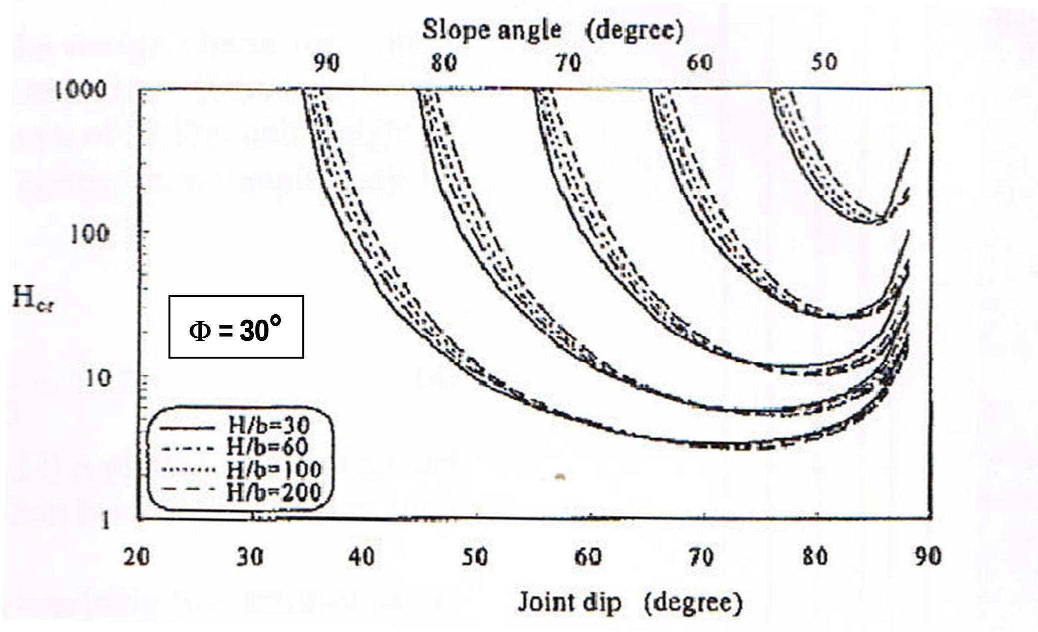


Figure 20, Flexural Toppling Design Chart for $\phi = 30°$, one of 7 such design charts produced by Adikhary, Dyskin, Jewell, and Stewart (1997); the others are for $\Phi = 10°$**, 15°, 20°, 25, 35°, and 40°.**

5. **Analysis of toppling mechanisms using numerical models:**

Toppling mechanisms are captured well in some two-dimensional numerical analyses. Many examples can be found in published literature and engineering reports, generally carried out with discontinuous deformation analysis (DDA) or the universal distinct element code (UDEC). Block toppling, which in general does not necessitate development and growth of new cracks, can be accurately captured by both DDA and UDEC models up to the point of crack development. Some examples are given in Figures 21 and 22. For a given sliding friction angle ($\phi_{allowable}$) of the discontinuities, the factor of safety of the modeled slope can be determined by progressive reduction of the friction angle input in the model until joint sliding initiates and column or blocks begin to rotate . The limiting friction angle found by this procedure is $\phi_{mobilized}$ and the factor of safety is (tan $\phi_{allowable}$ / tan $\phi_{mobilized}$).

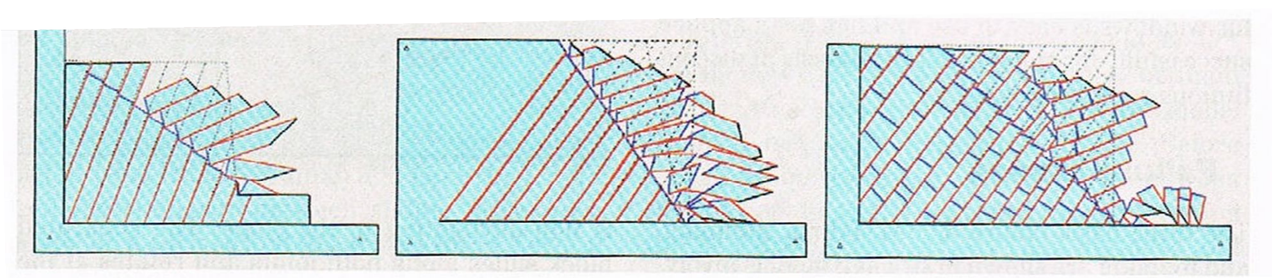


Figure 21 –DDA models of three different discontinuous rock masses, each with inclined toppling layers cut by one long, inclined joint. The ghost lines show the initial positions of layers and the initial slope surface. From Goricki and Goodman (2003)

The problem ceases to be compliant if block fracturing should initiate before the limit of equilibrium is reached. Figure 22 compares the results of a base friction model of a block slumping slope with its duplication by DDA. The upper-most surface block is subjected to bending and has cracked; this cracking was not, and could not be, captured by the numerical model.

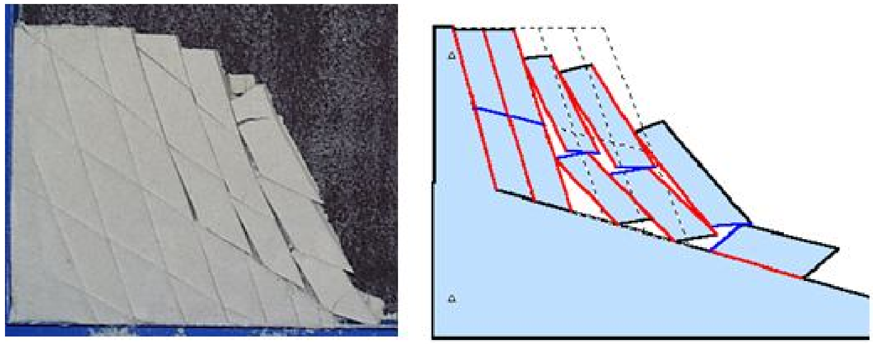


Figure 22 Block slumping failure captured in a base friction model, left, and a DDA model, right. From Goricki (1999)

Duplication of both fracture initiation and the development of failure is a significantly more complex objective for numerical modeling. A back-analysis of the results of Adikhary, Dyskin, Jewell and Stewart's centrifuge model studies of flexural toppling was reported by Alzo' Ubi, Martin, and Cruden (2010). These authors succeeded in duplicating the locations and style of cracking observed in the centrifuge experiments but only after introducing some imaginative tricks incorporated in "the UDEC Damage Model", inherited from experience attempting to model reinforced concrete beam s. Adikhary and Dyskin (2006) reported that finite element analysis using the Cosserat plasticity model, which incorporates rotations and bending moments in the constitutive relations, was able to reproduce displacements measured in new centrifuge experiments they conducted on toppling slopes. The Cosserat finite element analysis indicated that the mode of failure is sensitive to the friction angle, a relationship that was not incorporated in the preparation of the design charts previously discussed (see Figure 20) and which reduced their confidence in design charts corresponding to friction angles less than 15°.

6. A continuum of design approaches for excavating a safe slope in a rock mass capable of toppling

Assuming that a rock formation poses a general risk of toppling, one may choose between four approaches to the design and construction of an excavation, ordered according to the degree of disturbance and deformation that the rock mass will be permitted. Consider first the support of an underground gallery in horizontally layered rock (Figure 23).

1) If the roof is fragile, and there is considerable concern about the safety of mining, very stiff reaction logs can be pre-seated against the rock surface to disallow any deformation. This may be called ***under-pinning design.***

2) Or, modest supports can be installed such that the rock structure of the roof deforms only within the elastic limit, mobilizing supportive bending and compressive reaction forces within the rock. This may be called ***elastic design***.

3) A yet smaller support force permits additional internal rock-mass deformation which mobilizes friction forces on sliding surfaces. This may be called ***plastic design.***

4) If the initial support can not be placed to prevent destruction of the rock mass's own support capabilities, , an upper bound support force may be applied. In underground work this might be taken as a support force equal to the total weight of the roof rock. This assumes that the rock will have lost all internal support capability. We can refer to this approach as ***dead weight design.***

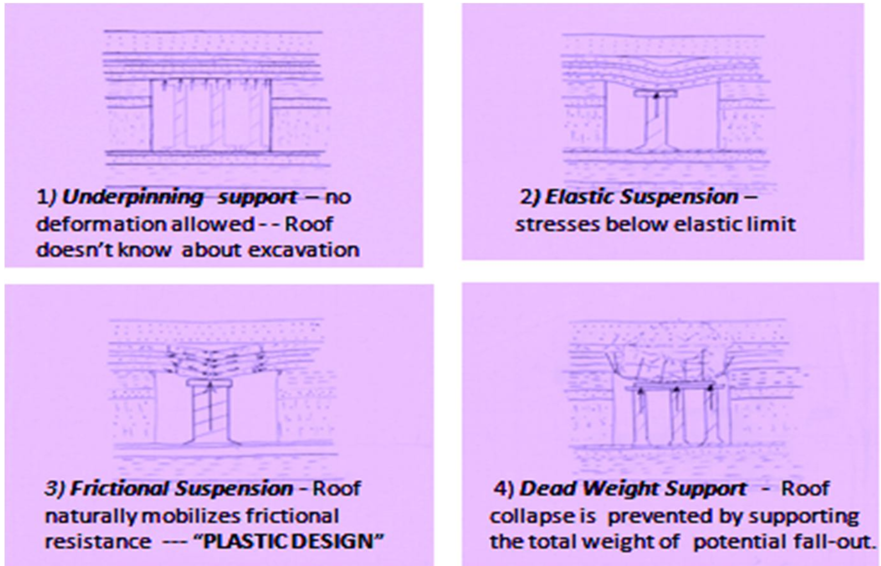
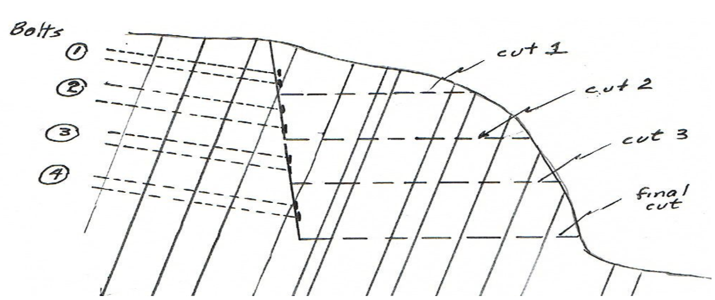


Figure 23. A continuum of four design philosophies, in order of increasing initial support force and permitted rock deformation for a hypothetical underground gallery in rock.

For designing and constructing an excavation into rock that exhibits geologic structure and/or behavior typical of toppling terrains , the most conservative approach would assume the rock to be broken by three or more sets of joints and consequently to design for block toppling. On the other hand, a rock mass with only one strong set of fractures, e.g. steeply dipping bedded rock with only widely spaced joints in other directions, may offer economies associated with a flexural toppling mechanism. The concepts of *underpinning, elastic design and plastic design* can be invoked to discuss how to proceed.

Underpinning design - In practice, it would be quite difficult, but not impossible, to realize an underpinning approach for a rock slope excavation. It could be attempted, for example, by counter-sinking re-minable bolts or cables extending continuously from the intended surface of the final cut slope to targeted locations of the anchors. If sufficient density and capacity of bolts were provided, and the ensuing excavation were carefully advanced , it is conceivable that deformation of the final cut line could be minimized sufficiently to preserve the integrity of the rock columns. In that case, the economies of flexural toppling could apply .[5]

Elastic design for an excavation in rock is attainable by installing sufficiently capable rock-bolts in each stage of a top-down excavation sequence, as depicted in Figure 24. Cut 1 is made and rock bolts are installed immediately. Then Cut 2 follows, the second stage of bolts are installed, etc. (Some method would probably need to be found for obtaining access to the bolts for measurements and adjustments as the excavation continues.) Analysis may help judge whether the procedure and supports are likely to have preserved the continuity of the joint columns.



[5] The author is not aware of cases where this procedure was applied in practice, except for an instance of countersinking large diameter rock bolts prior to excavation for repair of a damaged intersection of two large underground drifts, during construction of the Norad defense facility in Colorado.

Figure 24 - “Top down” excavation and support can be designed and executed to approach ideal elastic design. As long as there are no cross joints, the supports can be designed with the flexural toppling model.

Plastic design assumes that breakage of the columns may occur, or the rock is effectively already in a plastic state because cross-joints interrupt the continuity of the columns. In this case, one or more configurations of block columns can be studied , based on the geologic data, as sketched in Figure 25. The design must be developed to prevent the occurrence of block toppling in any of the statistically likely block configurations. Therefore it is economically advantageous to pursue careful field observations and exploration, including geophysical investigations, down hole televiewer logging, and/or downhole mapping by a geologist in large diameter borings.

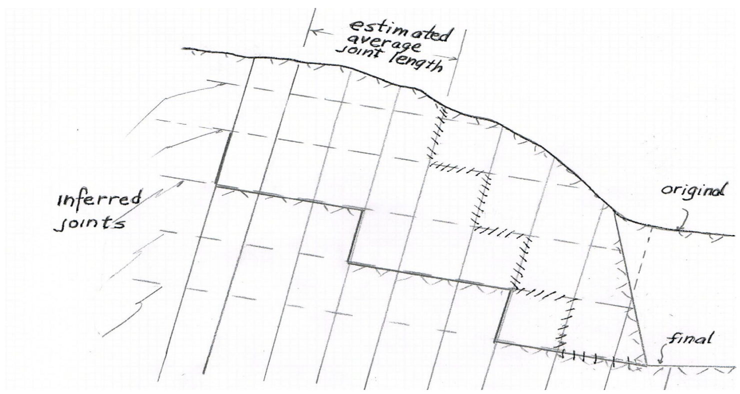


Figure 25 - Based on field studies, a variety of block models need to be analyzed for the plastic design

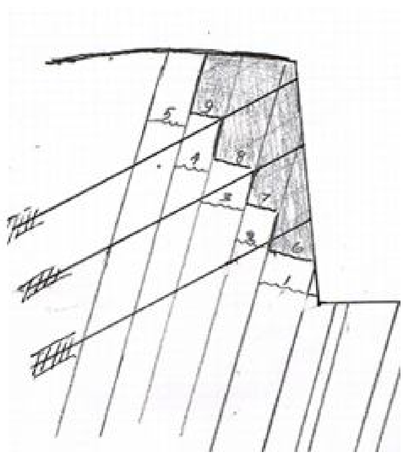


Figure 26 – One realization for the configuration of a plastic design. The tie-backs are selected and angled to assure safe anchorage and sufficient moment capacity.

“**Dead weight” design:** The notion of dead-weight design might conceivably be applied to a rock cut by assuming the rock had been reduced to a cohesionless soil. However, a failure in toppling can produce pockets of high-pressure water, large voids, and continuing hazards of falling rock, which could generate risks that a soil-based design would not normally face. Rock with a structure and composition prone to toppling can be considered to be a *sensitive* rock mass and should ideally be engineered to retain its inherited structure.

7. Some case histories of toppling in California

`Three examples, in which the author was involved as a consultant, will be outlined. They are: Caribou and Belden power projects of the *Pacific Gas and Electric Company,* on the North Fork of the Feather River; and correction , of a large landslide affecting a valuable Santa Barbara housing sub-division by the firm *Cotton-Shires & Associates.*

Bore hole extensometer response to toppling: Investigations in each of these cases included geologic mapping, subsurface exploration with deep drill-holes, , hydrologic studies, repeated surveying of multiple fixed surface points, and installation of deep, bore-hole extensometers. A slump, or a planar rock slide cutting through the line of the exten-someter can be recognized by a clear offset in the record. In contrast, active toppling is indicated by a continuously bending line line, with several wiggles, which represent secondary offsets resulting from flexural slip along rotating block sides.

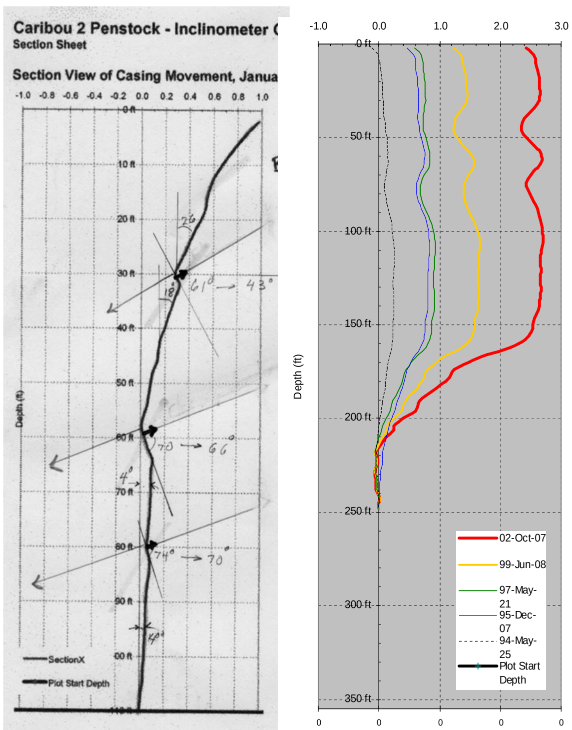
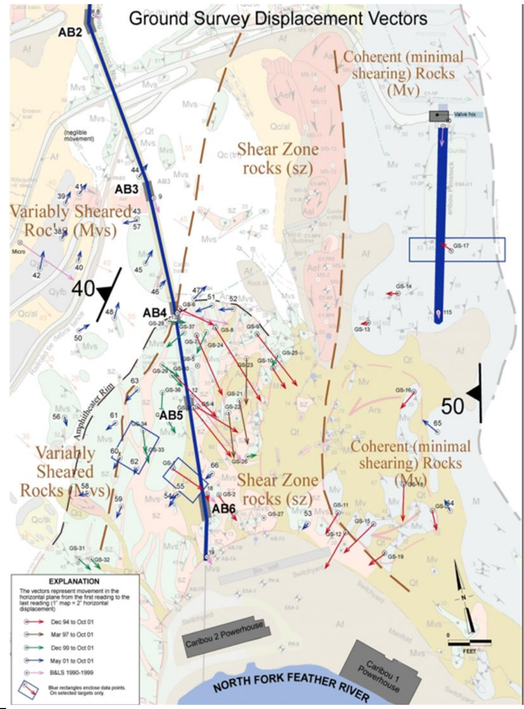


Figure 27 Extensometer records in boreholes cutting through
Toppling phyllite formations at Caribou (left) and Belden (right).

Caribou Penstock: The geologic setting of the Caribou project penstocks can be seen in Fig. 28a**.** The lower part of Penstock 2 (on the left) has been distressed by toppling in a direction transverse to the conduction. The arrows in Fig. 28b give the direction of movements of surface targets, as measured from the roof of powerhouse 1 (out of view on the bottom right). Note that the schistosity dips 40° into the steep hill west of penstock 2, opposite to the direction of movement. These deformations were exacerbated by the erosion of a steep-walled gully from concentrated runoff in the soft rock of the shear zone immediately to the east of penstock 2. The toppling motion has been slowed by installation of deep drains at the toe of the slope, and emplacement of measures to redirect and control runoff on the penstock slope.

a)



b)

Figure 28 - a) Aerial view of PG&E's Caribou penstocks #1 on the right and #2 on the left;
b)Geologic map by Dale Marcum of Cotton Shires & Assoc.

Belden Tunnel Cracking: Belden power tunnel receives Caribou discharge from a long tunnel, which daylights to cross the N. Fork of the Feather River Canyon in a siphon, and enters tunnel 2 in a side hill portal some 250 feet (80m) above the river. The portal region was excavagted in a highly fractured, weathered phyllite, with the foliation dipping gently into the hillside for the first 380 feet (115m) from the portal. There followed a 50 foot (15 m) zone of hard but highly fractured phyllite, and a vertically foliated, hard phyllite thereafter. Except for the circular steel lining extending 130 feet (39.3 m) from the portal, the tunnel is concrete-lined. It contains "weep holes" in the roof, intended by the designer to equalize internal and external water pressure in order to prevent collapse of the tunnel from external water pressure when dewatering.

Shortly after the initial operation of the facility in 1969, the system was dewatered and the tunnels were inspected, revealing a 0.5 inch (1.27 cm) open crack crossing the entire 15 foot (4.55 m) diameter, circular concrete lining (Figure 29a). The crack occurred 300 feet (130 m) into the tunnel, 80 feet (24 m) short of the transition between gently dippin, soft phyllite and the steeply dipping hard phyllite (Fig. 29b). For some time, the mechanisms causing the movement and cracking could only be conjectured. Possible hydraulic fracturing or jydraulic jacking of the tunnel rock was considered but *in-situ* tests reported a hydraulic jacking pressure value well above the tunnel water pressure. A valley-side slide was supposed, but no geomorphic evidence for that theory could be discovered.

Engineering geologic mapping and instrumentation confirmed that the rock mass in the downstream section of the tunnel was toppling. Figure 29b shows the three layers of a toppling mass (compare with Figs. 13 and 14), mapped from surface observations by Cotton Shires and confirmed by the response of borehole extensometers *I1 and I2.* Extensometers across the main cracks placed during an outage, and displacements surveyed at point A on the surface at the portal (Figure 29b)revealed similar rates of movement, which continued for 47 years until the problem was corrected by installing a waterproof PVC lining. Leakage from the cracks of the pressure tunnel was entering the rock mass along shear and foliation surfaces in the rock outside the tunnel, effectively jacking apart rock blocks of an ancient topple. These forces accumulated along the siphon (confirmed by closure measurements of the Dresser Couplings between the cans), and were ultimately reacted by a stiff dyke in the rock foundation at the foot of the slope. Thus the slope itself did not fail. Leakage through the weep holes probably contributed to the regeneration of toppling movements.

a) 

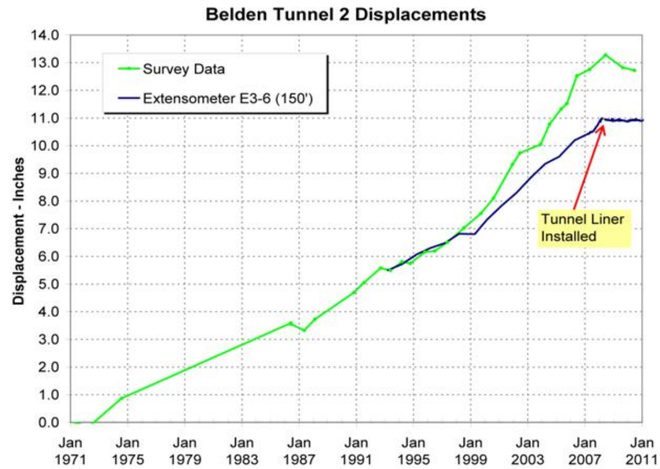
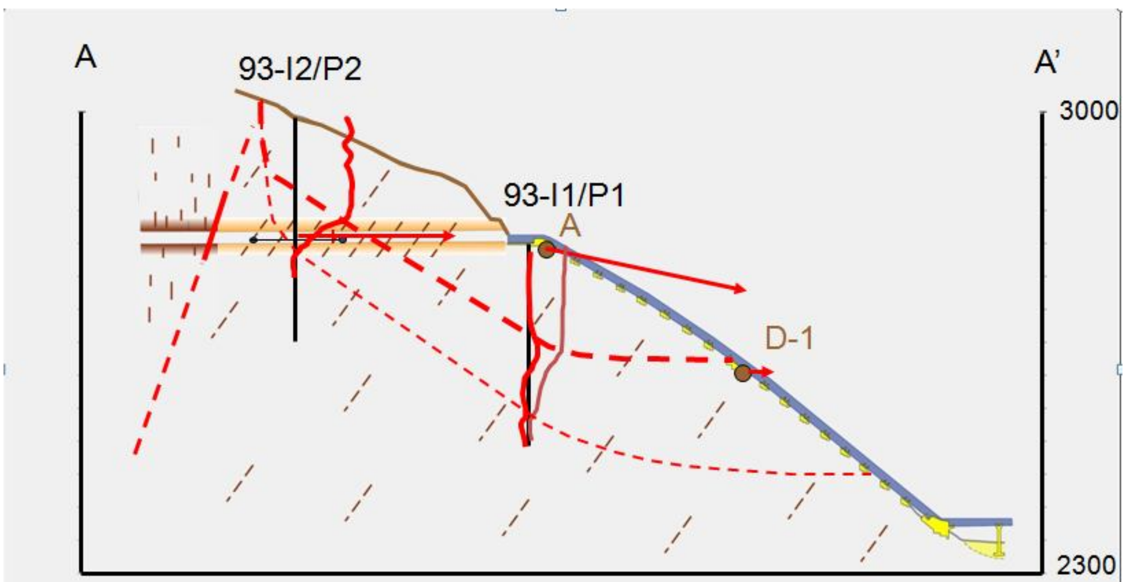


Fig. 29 One of the main cracks through the Belden Tunnel in 2006

Fig. 30 Measured displacements of the tunnel Up to January 2011, after PVC lining was placed.



Fig, 31 Belden tunnel instrument-response, super-imposed on the geologic section, shows

Consistent toppling movements in surface survey point A, and borehole extensometers P1 and P2. The dashed lines, mapped independently by Cotton Shires geologists conform to toppling layers 1, 2, and 3 described in Figs. 13 and 14.

Santa Barbara landslide: Excavation of a small quarry for road material at the foot of a housing sub-division near Santa Barbara, CA, triggered a slump in the Tertiary Rincon mudstone. The eastward progression of the soil-like failure of these soft rocks was parallel to the strike of the shale, which dips approximately 70° to the north. The steep northern side wall of the developing slide gully began to fail by toppling of overhanging mudstone block-columns. It was necessary, but not sufficient, to halt the progression of the slump to safeguard dwellings. The progression of the sidewall toppling failure was also endangering home foundations and infrastructure along the margin of the slide.

Fig. 32, by Cotton-Shires, shows a vertical section across the slide, looking approximately eastward up the main slide gully. The unstable, toppling zone behind the left gully wall was determined to be expanding, thus undermining an "incipiently toppling zone" behind it. Cotton-Shires determined to construct tied-back walls to apply resisting moment and halt this dangerous northward projection. The rock proved to be so weak that no significant advantage could be gained by a flexural toppling model. The calaculations of required tie-back tensions to deliver the required resisting moment were therefore based on block-toppling analyses using the spread sheet approach. Figure 33 shows an area destroyed by flexural slip with intense development of obsequent scarps.

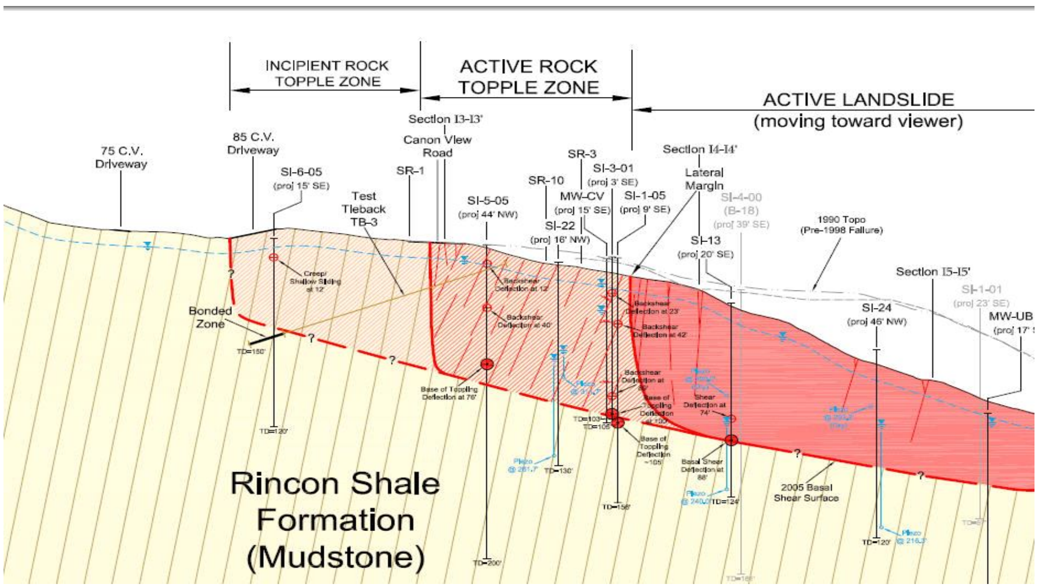


Figure 32 An east-west section of the Santa Barbara landslide looking upslope, opposite to the direction of slide motion. Drawing by Cotton-Shires.

Figure 33 – Severe damage from development of obsequent scarps, caused by toppling towards the left.

Figure 34 Construction of tied-back support walls designed by Cotton Shires.

Conclusion: The subject of toppling failure is not only interesting, but highly relevant to engineering geology, geomorphology, and rock mechanics. The paper has touched only upon civil engineering applications but there are surely other applications in mining, tunneling, and other fields as distant as farming, packaging, and container-handling. I wish to thank the Austrian Society for Geomechanics, and particularly Prof. Rainer Poisel, for giving me the privilege of preparing and presenting this lecture in the name of highly respected Professor Leopold Müller.

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