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BRITTLE CONTRACTIVE BEHAVIOR IN SOIL, TAILINGS, AND ROCK MASS

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ABSTRACT: The understanding of the mechanisms of loss of strength responsible for many significant slope failures has developed over recent years. Unforeseen slope failures, including tailings dam collapses, have been one of the main catalysts for the improvements to geotechnical understanding. Sudden loss of strength or brittleness can occur in the extreme case of these processes, such as liquefaction. The mechanism is typically understood to involve the rapid transition from drained to undrained behavior commonly associated with contractive behavior. Other slope failure mechanisms, including those that occur in rock masses, involve strength loss. Rock deformation has been previously characterized as dilative and therefore not likely to produce high water pressure associated with contractive deformation. However, contractive behavior can be shown to occur in rock mass kinematics such as toppling. In toppling, the layers or columnar blocks are initially in contact with each other and are in a configuration allowing down-slope rotation. The rate of movement is moderated by the frictional strength of the rock interfaces. Face-to-face contact between layers is lost as space opens up during rotation. This stage of failure can occur rapidly as the assemblage of layers loses its frictional strength and is effectively in a relatively free-falling condition. The layers may regain frictional strength once they have rotated into a condition of symmetry that restores the contact of rock interfaces. The new orientation of the toppled layers does not have the potential energy of the pre-toppling condition, and the slope may become locally self-stabilized. Between the initial and final conditions, there is a dilative and a contractive phase. The contraction of the rock mass has the potential to cause ejection of air and water, as is sometimes observed in rock slope failures. The contraction of the rock mass during deformation also has the potential to cause high water pressures, leading to a reduction in stability and a change to the failure mechanism.

Keywords: Brittleness, Contractive behavior, Strength loss, Soil, Tailings, Rock mass, Toppling

1. INTRODUCTION

It is understood that strength and strain are intimately related in geotechnics [1]. In particular, the strength of a material can change during strain. The reduction in strength, ‘strength loss’, being the most problematic for engineering purposes [1]. Loss of strength as a result of shearing has been described as brittleness since the 1940s [1]. The strength of soil at large strain conditions can be referred to as the residual strength [2]. The brittleness index, a ratio of peak minus residual strength divided by peak strength has been attributed to Bishop in the 1960s [2]. The brittleness index defined this way only measures the magnitude of the strength loss but the rate of reduction should also be considered part of the concept [2]. Brittleness is sometimes considered to apply to the drained condition of soil and has been distinguished from ‘sensitivity’ which occurs in the undrained condition [2]. According to [2], brittleness is a function of fabric change during shear whereas sensitivity is a result of pore pressure increase. In this paper the term brittleness is used broadly to refer to a wide range of strength loss conditions.

The history of the development of geotechnical knowledge has largely occurred when the strength of materials has been overestimated or the mechanisms of strength loss have not been fully appreciated and a catastrophic failure has unexpectedly occurred [3,4]. The examples in [4] show that rock mass, like soil, can undergo strength loss in an unexpected manner.

In the case of the Ontake landslide in Japan in 1984 an earthquake triggered a rock slide of over 30 million cubic meters with a run-out distance of over 5 km. The event was inferred to have been influenced by the preceding rainfall and movement focused on a weak pumice layer [6]. The Ontake landslide was included in a review of examples of entrainment of fluidized material contributing the long run-out distances in rock slope failures [7].

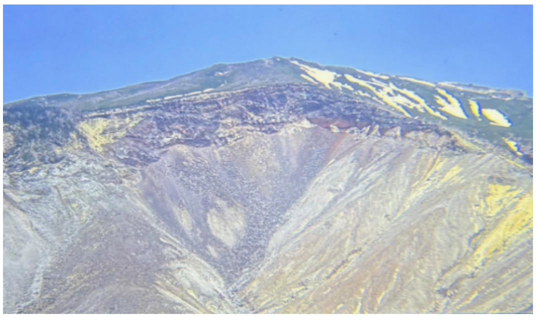

Fig.1 Ontake landslide scarp (photograph by the author, 1992)

2. RESEARCH SIGNIFICANCE

Brittleness, meaning rapid, significant strength loss can be considered a key concept across a wide range of geotechnical issues. Mechanisms of brittle strength loss are well known in soils and fine mine waste known as tailings. Understanding of brittle strength loss can be extended to rock mass conditions also. All these materials can have multiple strength conditions. If the conditions rapidly change from a stronger to a significantly weaker one then brittle strength loss has occurred. If an engineering design is based on a higher strength condition without regard to the potential transition to lower strength then serious consequences can occur. The overview of these concepts here is intended to draw attention to a range of brittle strength loss behaviors that can occur. The paper introduces the significance of contractive behavior to rock mass kinematics and dynamics.

3. CONCEPTS

Two main areas emphasized are firstly the importance of the potential for brittle strength loss in assessing the safety of tailings dams. Firstly, assessing strength loss is seen by many researchers and practitioners as more significant than determining the likelihood of a triggering event such as an earthquake. Secondly, toppling kinematics in rock slopes is addressed to illustrate brittle strength loss and contractive kinematics that can occur in a rock mass failure.

3.1 General

The stability of a slope is typically described in terms of factor of safety or probability of failure [8]. In order to calculate such numbers, the strength of the material must be understood in terms of its range of properties. Soil properties can be understood as drained versus undrained and as peak, fully softened and residual [8].

The terminology residual strength of soil needs to be differentiated from residual soil meaning a soil which has formed directly from the underlying rock without significant transportation [9]. These residual soils are typically quite dense as they are undisturbed by transportation [9]. Their behavior is typically comparable to over consolidated soils [9].

Experimental laboratory studies can be conducted to assess the peak and residual shear strength of various material compositions and moisture conditions [10]. The strength of cohesive and granular soils typically increases with depth [11]. Specifically, the strength of soils is influenced directly by the confinement condition [11]. Various studies have explored the relationships of plasticity via the ‘velocity field’ or kinematics of the deforming system in relation to the stress distribution. These models can be applied to common problems such as loading of soft soils by embankments or other earthworks [11].

Strain softening behavior of soil and rock can be observed in laboratory experiments and applied in geotechnical analysis [12]. Such behavior has been observed at low confinement conditions where the rate and magnitude of the reduction in strength can be most pronounced [12]. A plastic shear strain threshold was applied to define the transition from a linearly decreasing strength with strain from a constant strength (residual) condition at greater strain [12].

3.2 Brittleness

The concept of brittleness and the brittleness index can express the sudden loss of strength of soils and other granular materials and has been used since the 1980s and earlier [13]. The original definition of brittleness was based solely on the magnitude of the strength reduction. However, the commonly used meaning of the term ‘brittle’ includes a sense of rapid loss of strength. One approach to quantifying degrees of brittleness and the relationship to strain has been developed for tailings [14]. In this scheme, non-brittle or moderately brittle materials do not undergo any reduction in shear strength until 3% axial strain is developed (Fig.2).

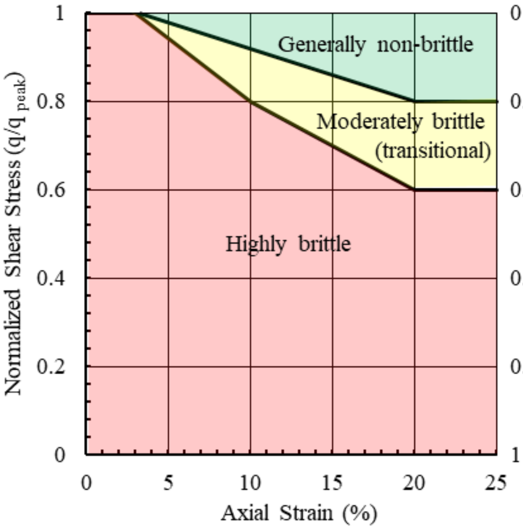

Fig.2 Definition of brittleness developed for tailings materials [14]

Non-brittle and moderately brittle materials can undergo 20% and 40% strength reduction, respectively. However, if these strength reductions occur prior to development of 20% axial strain then the materials may qualify for a higher degree of brittleness in this scheme (Fig.2).

3.3 Seismicity

Earthquakes stand out as a natural phenomenon with potential to cause destruction and damage through their effects on structures. A seminal work by the US Corps of Engineers (1980's) [15] outlined a pseudo-static method by stating "the seismic stability of embankment dams may be evaluated by a relatively simple method … in cases where there is no threat of liquefaction or severe loss of shear strength under seismic shaking.". That study emphasized that the pseudo-static approach to earthquake impact on dams is a 'screening tool' meaning it is not a sufficient check on stability itself but can be used to classify levels of concern that lead to applying appropriate acceptance criteria [15].

That work emphasized that dams with embankments or foundations that are susceptible to liquefaction or significant softening by cyclic loading require more specific stability assessments than can be achieved by pseudo-static approaches [15]. That work also referred to the "dynamic yield strength" of many earth materials being approximately 80% (i.e. 20% strength reduction) of the pre-seismic condition and that such strength reduction in analysis can be an appropriate way to assess seismic/cyclic conditions [15].

Further developments in seismic responses of dams in the 1990's [16] considered the strength of liquefied materials and noted they have a 'steady-state' or 'residual' strength in the liquefied condition. Those studies found that triggering of failure in laboratory testing occurred at "the point at which control of the test is lost: if one were to stop the test in a load-controlled fashion after sufficient strain (with the stresses still applied) the sample would creep and ultimately flow in the steady-state mode" [16]. The residual strength of liquefied material is typically given as a strength to stress (i.e. depth) ratio [16].

3.4 Dilation and Contraction

The behavior of granular material is dependent on the density which is commonly measured as void ratio [16]. During shearing, granular materials will contract if they are loose or dilate if they are dense relative to a critical void ratio that represents a condition of shearing with constant volume. The critical void ratio varies with the degree of confining stress [16]. The potential for granular material to be contractive has been recognized as a key consideration in stability of tailings impoundments [14].

A mechanism called 'void redistribution' has been proposed as an explanation for the time delay that has been observed between possible triggers and actual slope failure [17,18]. The typical void redistribution scenario can occur where granular material is below an impermeable layer. The trigger event causes some contraction of the granular material which expels excess water upward. However, the upward progress of water is prevented by the presence of the impermeable layer above [17,18]. In an extreme case, a film of water forms at the base of the impermeable layer with detrimental effects on stability of sloping ground. This mechanism is postulated in some rapid and significant loss of strength events [17]. It is understood that the void redistribution process takes time, leading to a delay between trigger and failure [17]. The process of void redistribution has been verified by centrifuge tests by various authors as reviewed by [17].

3.5 Strain Compatibility

Strain compatibility refers to the general concept that where different materials are in contact they may undergo different strains or strain rates in response to the stress conditions. Consequently, local stress variation or disturbances to the velocity field can cause strain incompatibility and can lead to discontinuity formation. Strain compatibility is also related to strain localization during shearing as the emerging shear zone must be oriented in a direction of zero longitudinal strain to maintain strain compatibility between the sheared and unsheared material [19]. These lines of no longitudinal strain are also known as the characteristic lines and at 90° to each other in conditions of constant volume but at less than 90° to each other in conditions of dilation [19].

4. SOIL AND LANDSLIDES

Soils are typically derived from natural rock weathering either as a residuum or by transportation. Soil behavior is strongly influenced by the interaction of clay and water [8]. The 'fully softened' condition indicates that the shear strength of clay-dominated materials moves toward a low strength condition through the absorption of water over time, especially concurrent with strain [8].

The ability of pore water to migrate through a soil under stress defines the undrained and drained strength conditions. Undrained strength is typically less than the drained strength but this relationship can be reversed over parts of the confining stress range, particularly in over-consolidated conditions [20]. Pressure changes in the water within the pores of soils are intimately related to strain and thereby control the soil strength via the effective stress principle. Negative pore pressures can develop in unsaturated soils and when lost by saturation, can result in rapid loss of strength [8].

Non-Newtonian or thixotropic behavior and creep can affect the stress-strain relationships in a wide range of materials [21-25]. Other processes that can influence soil behavior include progressive changes to grain-size through particle crushing and the discontinuous deformation of fissured clays [3].

Analysis of a faulted coarse gravel deposit in New Zealand showed a reduction from 78% clasts to 33% clasts within the fault zone. The process of faulting in this gravel deposit occurred by dilation of the clast-supported fabric to facilitate movement and concurrently for clay matrix material to migrate to the fault zone. This process is representative of inter-particle dilation to a critical state as is required for shearing of particulate systems generally. Due to the coarse grain-size of the gravel the changes caused by dilation can be observed directly in the field [25] (Fig.3).

Fig.3 Dilation of a fault zone in a gravel deposit observed by reduced clast concentration [25]

5. TAILINGS

Tailings are the end product of rock and mineral grinding in mineral processing. Many mines place these tailings in impoundments by hydraulic methods [26]. A number of high profile tailings impoundment failures have occurred causing significant loss of life and environmental impact [27]. Selected significant failure events are only briefly described here and readers should address the extensive literature for additional data and interpretation.

Various events and their analysis have been used in developing international [14] and national guidelines [28]. According to these guidelines, limit-equilibrium analysis remains the main tool for assessment. A factor of safety of 1.5 has been used for static conditions and 1.1 or 1.0 for seismic conditions [28]. These guidelines also indicate that seismic assessment must be based on cyclic and post-liquefaction parameter reduction rather than relying on pseudo-static analysis [14,28].

One of the well studied events was the of failure of the Mount Polley tailings storage facility in British Columbia, Canada in 2014. The failure mechanism was found to be complex and involved strain weakening in the foundation and its strain incompatibility relative to the overlying rockfill embankment [29].

The Cadia mine in Australia had a wall failure of a tailings storage facility in 2018. The investigation found a major cause of failure was a previously unknown layer of intensely weathered soil in the dam foundation [30]. The review stated that “This material, which had not been previously identified, is relatively weak, highly compressible, and strain-weakening (brittle) when subjected to load.” [30].

A significant development has been the increasing recognition that critical state soil mechanics is fundamental to tailings behavior [31,32]. The materials of the foundation, embankment and tailings can be defined relative to the critical state line in void ratio terms. This value is the state parameter and is positive for materials loose of critical state and negative for materials dense of the critical state [31,32]. The relationship between strength and confinement is related to the important concepts of dilation and contraction [33]. The value used for the boundary between dilative (non-liquefiable) and contractive (liquefiable) is -0.05 [33]. This value recognizes that materials just dense of the critical state can undergo liquefaction-like flow [33].

While limit-equilibrium is currently the prime method of analysis it has been recognized that limit-analysis applied in a finite element formulation can overcome some limitations of limit-equilibrium in the area of slip surface identification [34]. In limit-analysis the lower bound is based on an admissible stress field and the upper bound is based on a kinematically admissible velocity (plastic strain rate) field [34]. Limit-analysis and finite element analysis generally can also complement limit-equilibrium by responding to latent instability to undrained failure and stress rotation causing undrained failure [33,34].

Another aspect of tailings impoundment failure that is not covered here is dam breach. Tailings dam breach outflow is a highly complex process and is critical to assessing potential impacts [35]. In dam breach events water ponded on the tailings surface erodes tailings and re-hydrates it to a more flowable form even beyond the effects of tailings liquefaction with similarities to the entrainment effects of soil slope failures with non-Newtonian behaviors [7].

6. ROCK MASS

Rock material is typically brittle in nature and at high deviatoric stress can undergo microscopic damage starting at about 40% of uniaxial compressive strength [36]. Rock mass refers to the rock material and the naturally occurring fractures that are typically present. These systems of fractures are the main control on how the rock mass behaves [36]. In order for a rock mass to deform the confinement is typically low such that the blocks of rock can move relative to each other. The extreme example of low confinement is where fractures daylight in a face making the sliding or rotating motions of blocks feasible [37].

6.1 Toppling

Where rock mass deforms mainly by the rotation of elongate blocks, toppling is the failure mechanism [37]. Free-fall toppling where columns or layers of rock at a slope face rotate under the influence of gravity is a very recognized type of toppling. Toppling also occurs where the adjacent blocks form a confining condition which can influence the rotation [38]. This type of toppling is similar to the processes observed in hillside creep of layered bedrock [38]. Initially, the layers must overcome the interface friction, however, the frictional strength is quickly lost as the interfaces separate from each other [38] (Fig.4).

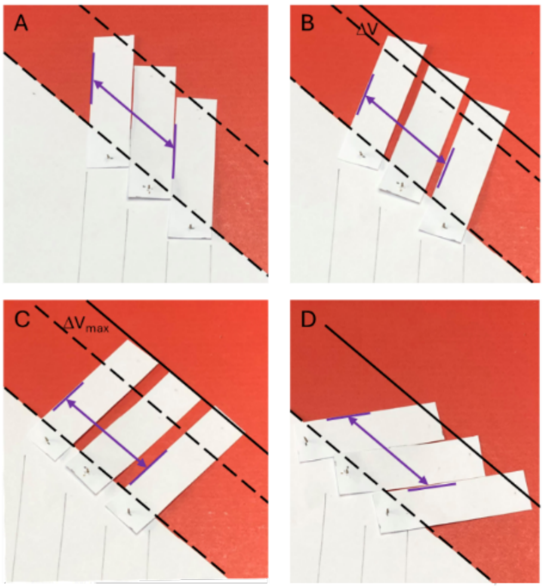

Fig.4 Toppling dilatancy in a paper model. A) Initial configuration. B) Opening phase. C) Maximum separation of blocks. D) Closure (self-stabilization)

Toppling of columns or layers within a slope has a dilative and contractive phase (Fig.4). The dilative phase develops until the layers are perpendicular to the basal surface of rotation. The contractive phase then continues until the layers reach a mirror image configuration compared to their starting postion. At this point the toppling can self-stabilize. During the dilative phase the frictional contact between adjacent blocks decreases and the rock mass strength decreases.

Therefore, the toppling mechanism can be considered brittle in the sense that strength decreases as strain is accumulated in the early phase of deformation. Examples of toppled slopes in layered rocks are shown in Figure 5 and 6. The layers have buckled (Fig. 5B, Bk) where the layers have rotated to be perpendicular to the basal surface. Separation of layers in this zone also shows the occurrence of local dilation (Fig. 5B, D). Layers of different thickness can respond differently by having different angles of rotation compensated for by shearing in the same sense as the downslope movement (Fig. 5B, location 1 and 2 and shear arrows).

Physical models [39] show the evolution of spaces between toppling layers. These spaces can progressively open and close as shown in Fig. 6. Figure 6 has been simplified to show alternate layers only. If water is present in these spaces during the closing phase then high water pressures could occur exacerbating the rock mass failure. This is in contrast to the assumption of previous studies [7] that "…volume increase negates nearly all possibility of a fluid pore pressure existing in the fragmented rock itself, at least in the initial stages of motion". Interestingly, some reports of rock slides have described 'geyser-like' water fountains occuring early in the failure initiation [40,41].

7. CONCLUSIONS

Geotechnical engineering for foundations, excavations and earthworks is fundamentally based on determining the strength of earth materials. These materials include soils and rock and also industrial waste products such as the finely ground rock residue known as tailings. The strength of these materials can be determined by field and laboratory tests. However, the strength can change under a wide range of conditions including saturation by water, loading and strain. When strength changes to a lower value this can be considered to be a strength loss. If the strength loss is unexpected then failure may be the result. The strength of geological materials including soils, tailings and rock masses is intimately associated with the amount, rate and type of strain that occurs. In particular 'loss of strength' that is sudden or unexpected reduction in strength of these materials should be considered along with the strain occurring during slope deformation.

The case of rock toppling can be included in brittle behavior in that the gravitational force induces shear strain in a downslope direction. If this shear strain induces rotation of elongate blocks of rock the kinematics of this proces can cause dilation and contraction of the deforming zone occuring in concert with separation of the side faces of the elongate blocks. If this toppling occurs within a confined condition such as hillslope creep or deep-seated toppling then complex internal kinematics can be observed or anticipated within the toppling rock mass.

These processes introduce mechanisms of deformation which would not occur without the dilation of the rock mass driven by the rotation of elongate blocks. Therefore, this pattern of strain (kinematics) leads to a reduction in strength. The rotation of the elongate blocks can eventually lead to a more stable configuration as the blocks attain an orientation equal and opposite to the original configuartion relative to the basal plane of the toppling. This stabilising configuration has been referred to as the mirror image configuration that can lead to self-stabilization of toppling [38]. The significant contribution here is the contractive phase of the deformation which has the potential to cause local increases in water pressure. Just as pore water pressure changes affect soil strength so to it is anticipated that the closure of rock voids during deformation would leads to reduction in the rock mass strength.

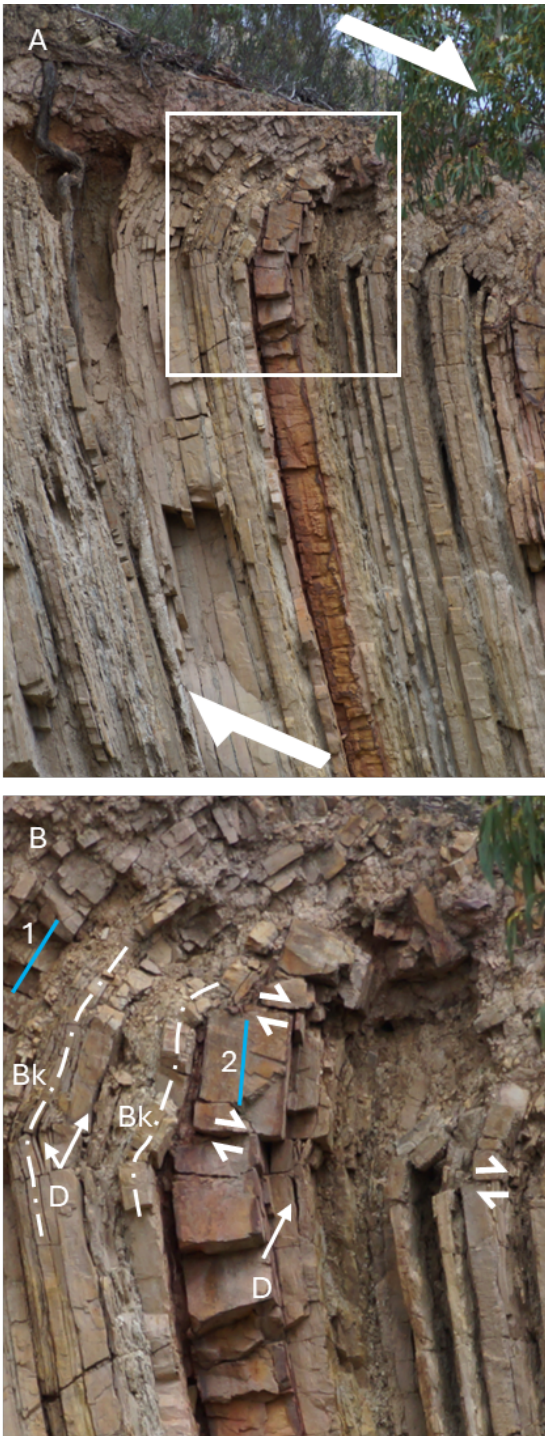

Fig.5 Constrained toppling showing the complex shearing and rotation accommodated in the dilating zone (Melbourne Formation, Plenty Gorge, Victoria Australia). A) Large arrows show the gravity-driven shear sense. B) Detail with features described in the text

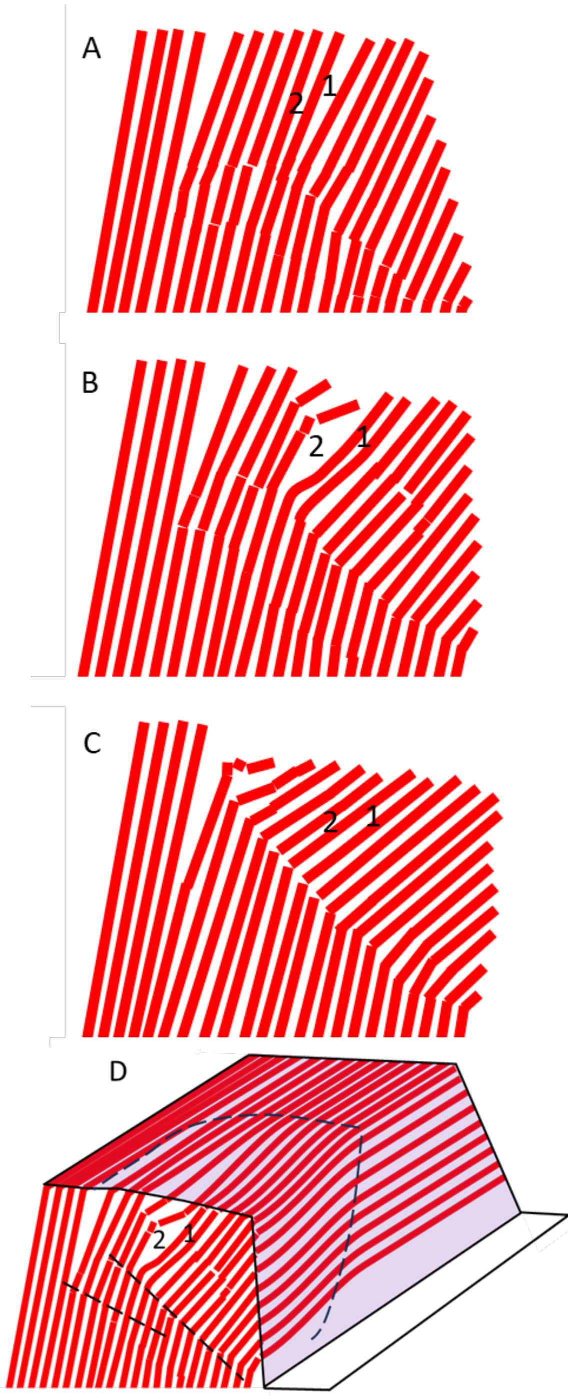

Fig.6 A-C) Evolution of dilation and contraction in toppled strata in physical modeling redrawn and simplified from [39]. Numbers 1 and 2 represent interfaces which open and close. D) A conceptual three-dimensional representation of the middle stage.

8. ACKNOWLEDGMENTS

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9. REFERENCES

[1] Bromhead E. N., Reflections on Slope Stability Engineering. CRC Press, 2024, pp.1-182.

[2] Bromhead E.N., Reflections on the Residual Strength of Clay Soils, With Special Reference to Bedding-Controlled Landslides. Quarterly Journal of Engineering Geology and Hydrogeology, Vol. 46, Issue 2, 2013, pp.132-155.

[3] Duncan J. M., Wright S. G., and Brandon, T. L., Soil Strength and Slope Stability, John Wiley and Sons., 2014, pp.1-333.

[4] Sonnekus M., and Smith J. V., A Review of Selected Unexpected Large Slope Failures. International Journal of GEOMATE, Vol. 15, Issue 48, 2018, pp.66-73.

[5] Guthrie R. H., Evans S. G., Catane S. G., Zarco M. A., and Saturay R. M., The 17 February 2006 Rock Slide-Debris Avalanche at Guinsaugon Philippines: a Synthesis. Bulletin of Engineering Geology and the Environment, Vol. 68, 2009, pp.201-213.

[6] Voight B. and Sousa J., Lessons from Ontake-san: a Comparative Analysis of Debris Avalanche Dynamics. Engineering Geology, Vol. 38, Issue 3, 1994, pp.261-297.

[7] Hungr, O., and Evans, S.G., Entrainment of Debris in Rock Avalanches: an Analysis of a Long Run-Out Mechanism. Geological Society of America Bulletin, Vol. 116, Issue 9-10, 2004, pp.1240-1252.

[8] Idries A., and Stark T. D., Uncertainty in Drained Fully Softened and Residual Strength Correlations. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 150, Issue 1, 2024, pp.6023010.

[9] Wesley L. D., Geotechnical Engineering in Residual Soils. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris, John Wiley and Sons. 2010, pp.3479-3482.

[10] Rusdiansyah A., and Barkiah I., The Behavior of Residual Shear Strength of Laterite Soil Due to the Addition of Coarse Sand and Low Plasticity Clay Fractions. International Journal of GEOMATE, Vol. 21, Issue 86, 2021, pp.100-107.

[11] Davis E. H., and Booker J. R., The Effect of Increasing Strength with Depth on the Bearing Capacity of Clays, Geotechnique, Vol. 23, Issue 4, 1973, pp.551-563.

[12] Renani H. R., and Martin C. D., Factor of Safety of Strain-Softening Slopes, Journal of Rock Mechanics and Geotechnical Engineering, Vol. 12, Issue 3, 2020, pp.473-483.

[13] Highter W.H. and Tobin R.F., Flow Slides and the Undrained Brittleness Index of Some Mine Tailings, Engineering Geology, Vol. 16, Issue 1, 1980, pp.71-82.

[14] International Commission on Large Dams (ICOLD). Tailings Dam Safety Bulletin N° 194, Version 1.0, Paris, Tailings Dams and Waste Lagoons, 2025, pp.1-308.

[15] Hynes M. E., and Franklin A. G., Rationalizing the Seismic Coefficient Method, USCE, GL-84-13, 1984, pp 1-21.

[16] Marcuson W. F., Hynes M. E., and Franklin A. G., Evaluation and Use of Residual Strength in Seismic Safety Analysis of Embankments, Earthquake Spectra, Vol. 6, Issue 3, 1990, pp.529-572.

[17] Kulasingam R., Malvick E. J., Boulanger R. W., and Kutter B. L., Strength Loss and Localization at Silt Interlayers in Slopes of Liquefied Sand, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 130, Issue 11, 2004, pp.1192-1202.

[18] Kokusho T., and Kabasawa K., Effect of Void Redistribution or Water Films on Shear Strength of Liquefied Deposits, In, Geomechanics: Testing, Modeling, and Simulation, American Society of Civil Engineers (ASCE) 2005, pp. 475-503.

[19] Smith J. V., and Durney D. W., Experimental Formation of Brittle Structural Assemblages in Oblique Divergence, Tectonophysics, Vol. 216, Issue 3, 1992, pp.235-253.

[20] Shewbridge S., Undrained Strengths and Long-Term Stability of Slopes, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 145, Issue 11, 2019, pp. 2519002.

[21] Haghighat E., Rassouli F. S., Zoback M. D., and Juanes R. A Viscoplastic Model of Creep in Shale, Geophysics, Vol. 85, Issue 3, 2020, MR155-MR166.

[22] Jiang H., and Jiang A., Numerical Analysis and Application of Nishihara Creep Nonlinear Damage Model Considering Ubiquitous Joints, Tunnelling and Underground Space Technology, Vol. 157, 2025, pp.106335.

[23] Bougouin A., Metzger B., Forterre Y., Boustingorry P., and Lhuissier H., A Frictional Soliton Controls the Resistance Law of Shear-Thickening Suspensions in Pipes. Proceedings of the National Academy of Sciences, Vol. 121, Issue 17, 2024, pp. 2321581121.

[24] Smith J. V., Shear Thickening Dilatancy in Crystal-Rich Flows, Journal of Volcanology and Geothermal Research, Vol. 79, Issue 1, 1997, pp. 1-8.

[25] Smith J. V., Dilatancy Fabrics in Conglomerates in the Awatere Fault Zone, New Zealand, Journal of Structural Geology, Vol. 20, Issue 11, 1998, pp.1449-1454.

[26] Blight G. E., Geotechnical Engineering for Mine

Waste Storage Facilities. CRC Press, 2009, pp.1-634.

[27] Blight G. E., and Fourie A. B., Catastrophe Revisited–Disastrous Flow Failures of Mine and Municipal Solid Waste, Geotechnical and Geological Engineering, Vol. 23, 2005, pp.219-248.

[28] Australian National Committee on Large Dams (ANCOLD), Guidelines on Tailings Dams: Planning, Design, Construction, Operation and Closure, 2019, pp.1-79.

[29] Zabolotnii E., Morgenstern N. R., and Wilson G. W., Mechanism of Failure of the Mount Polley Tailings Storage Facility. Canadian Geotechnical Journal, Vol. 59, Issue 8, 2022, pp.1503-1518.

[30] Jefferies M., Morgenstern N. R., Van Zyl D., and Wates J. Report on NTSF Embankment Failure Cadia Valley Operations for Ashurst Australia. Independent Technical Review Board, 2019, pp.101.

[31] Been K., Characterizing Mine Tailings for Geotechnical Design, Australian Geomechanics Journal, Vol. 50, Issue 4, 2016, pp. 59-78.

[32] Jefferies M., On the Fundamental Nature of the State Parameter, Géotechnique, Vol. 72, Issue 12, 2022, pp.1082-1091.

[33] Reid D., Fanni R., and Fourie A. B., Assessing the Brittleness of Tailings–Some Common Pitfalls and Uncertainties. Dams for People, Water and Environment and Development - 92nd ICOLD Annual Meeting New Delhi, India, 2024.

[34] Whittle A. J., El-Naggar H. M., Akl S. A., and Galaa A. M., Stability Analysis of Upstream Tailings Dam Using Numerical Limit Analyses, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 148, Issue 6, 2022, pp.4022035.

[35] Sreekumar U., Gildeh H. K., Mohammadian A., Rennie C., and Nistor I., Tailings Dam Breach Outflow Modelling: A Review. Mine Water and the Environment, 2024, pp.1-25.

[36] Hoek E., and Brown E. T., Practical Estimates of Rock Mass Strength, International Journal of Rock Mechanics and Mining Sciences, Vol. 34, Issue 8, 1997, pp.1165–1186.

[37] Martin C. D., Alzo'ubi A. K., and Cruden D., Progressive Failure Mechanisms in a Slope Prone to Toppling, In Proceedings of the International Symposium on Rock Slope Stability in Open Pit Mining and Civil Engineering, Vancouver, Canada, Vol. 1, 2011, pp. 1-12.

[38] Smith J. V., Self-Stabilization of Toppling and Hillside Creep in Layered Rocks. Engineering Geology, Vol. 196, 2015, pp.139-149.

[39] Ning Y., Shen Y., Ding T., Xu P., Duan F., Zhang B., Zhang B., and Smith J.V., 2025. Evolution Process of Toppling Deformations in Interbedded Anti-Inclined Rock Slopes. Applied Sciences, Vol. 15, Issue 14, 2025, pp.7727.

[40] Walter F., Amann F., Kos A., Kenner R., Phillips, M., de Preux A., Huss M., Tognacca C., Clinton J., Diehl T., and Bonanomi Y., Direct Observations of a Three Million Cubic Meter Rock-Slope Collapse with Almost Immediate Initiation of Ensuing Debris Flows. Geomorphology, Vol. 351, 2020, pp.106933.

[41] Friele P., Millard T.H., Mitchell A., Allstadt K.E., Menounos B., Geertsema M., and Clague J.J., Observations on the May 2019 Joffre Peak Landslides, British Columbia. Landslides Vol. 17, Issue. 4, 2020, pp.913-930.

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