WEATHERING AND EROSION PROCESSES IN ROCKS – IMPLICATIONS FOR GEOTECHNICAL ENGINEERING

Steve Hencher
Halcrow China Ltd

Abstract: *This paper provides an introduction into active processes of weathering and how these affect the engineering properties of rocks. Five particular aspects are dealt with in some detail. The first concerns the shear strength of intact materials with particular consideration of the transition from rock-like to soil-like behaviour. The second topic is permeability of weathered rock profiles and in particular the often channelised flow of groundwater. Topic three concerns material disintegration and the growth of joints in the weathered rock mass; a classification of joints based on their "life-cycle" from oriented micro fractures to fully developed planes of weakness is introduced. The fourth topic concerns weathering along joints and the consequences for shear strength of such discontinuities. Finally deterioration of the overall rock mass is considered in a geomorphologic development sense, where such deterioration occurs and how such deterioration may be taken as indicative that failure is developing.*

INTRODUCTION

This paper is a written version of a talk presented at the Symposium on Hong Kong Soils and Rocks held in March 2004. The paper follows the structure of presentation which addressed a series of topics from the author's knowledge and experience. No in-depth review was attempted for each topic at the Symposium and that is also the case for this paper.

The paper provides an introduction to weathering processes, weathered profile characteristics and engineering consequences together with discussion on five topics that the author considers are generally poorly understood or poorly documented. The five specific topics addressed in some detail are as follows:

1. Decomposition and loss of shear strength
2. Changes in permeability
3. Disintegration and the growth of fractures
4. Weathering of rock joints and consequences for shear strength
5. Deterioration and progressive collapse

INTRODUCTION TO WEATHERING PROCESSES AND ENGINEERING DETERIORATION

Rocks in Hong Kong show significant affects of weathering down to depths of often tens of metres and occasionally over 100 metres. Weathering is manifested by changes from the original rock state (fresh) including changes in mineralogy, colour, degree of fracturing, porosity and thereby, density, strength, compressibility and permeability. The main rocks encountered in Hong Kong are Jurassic and Cretaceous-aged volcanic extrusive/sedimentary sequences and granitic intrusions; it is the weathering of these rocks that is the subject of much of this paper. Other, rarer, rocks including meta-sediments and karstic limestone do occur in Hong Kong and cause particular problems, especially for foundations, but the

Figure 1 Iron and manganese oxide staining on joints

singular weathering characteristics of these rocks are not considered here. That said, many of the concepts such as joint development and progressive deterioration might be taken as generic and not peculiar to the granites and volcanic rocks of Hong Kong.

Weathering Profiles

As a general rule, weathering works in from free surfaces where chemicals in water (including the water itself) can attack the parent rock. Weathering processes generally operate above the water table in the *vadose* zone although it should be noted that water tables change periodically and certainly are likely to do so over the millions of years that it may take for thick weathered profiles to develop. Therefore current water levels may not be related to depth of weathering.

Figure 1 shows discoloration due to mineral decomposition and deposition associated with localised flow along rock joints. It should be recognized when trying to understand weathering development or to generalise about hydraulic conductivity that water paths through the rock mass and even along a single joint are usually tortuous (Kikuchi and Mito, 1993). This makes interpretation of water conductivity tests open to various interpretations (Black, 1990). Channel flow governs the movement of groundwater in fractured rock and fracture networks develop with time, as discussed later in this paper.

At any particular location the weathering profile is a function of parent geology, groundwater conditions and the relatively recent geological and geomorphologic history of the site. Profiles may be ancient and may bear little relationship to the current geomorphologic setting.

The very rapidly changing conditions at the surface of the Earth (in geological terms) need to be appreciated. Only ten thousand years ago the sea level was perhaps 100 metres lower than it is today (Pirazzoli, 1996) and the effect that such a change in base levels would have on down-cutting rates of rivers can be readily appreciated. When the sea level was so low, many other parts of the world were covered by glaciers. In Hong Kong, climatic conditions were probably quite different to those pertaining today.

Given these and other factors, weathering profiles can be rather unpredictable from examination of the current topography alone.

For example, whilst valleys can be associated with thick saprolite where weathering has preferentially attacked fractured rock associated with faulting (e.g. Shaw and Owen, 2000), at many other locations the valley floor lies directly on rock. At such locations erosion has and

continues to remove weathered products faster than decomposition can take place. Conversely, hill tops can comprise rock with no weathering cover (e.g. Lion Rock), but elsewhere hills can comprise thick saprolite, for example at Tung Chung East (Halcrow Asia Partnership, 2000). Even major faults with mappable displacements are not always associated with deep weathering.

Weathering processes and the resulting, sometimes complex, weathered profiles is the subject of many excellent textbooks (e.g. Ollier, 1984) and papers and it is not intended to try to summarise those findings in any detail here. The wide varieties of conditions that can be encountered in weathered terrain were illustrated schematically by Ruxton and Berry (1957) and Figure 2 is based on their interpretation of weathered profiles in Hong Kong. The important concept is that weathering profiles are not fixed in time and that different degrees of maturity of profile can be recognized.

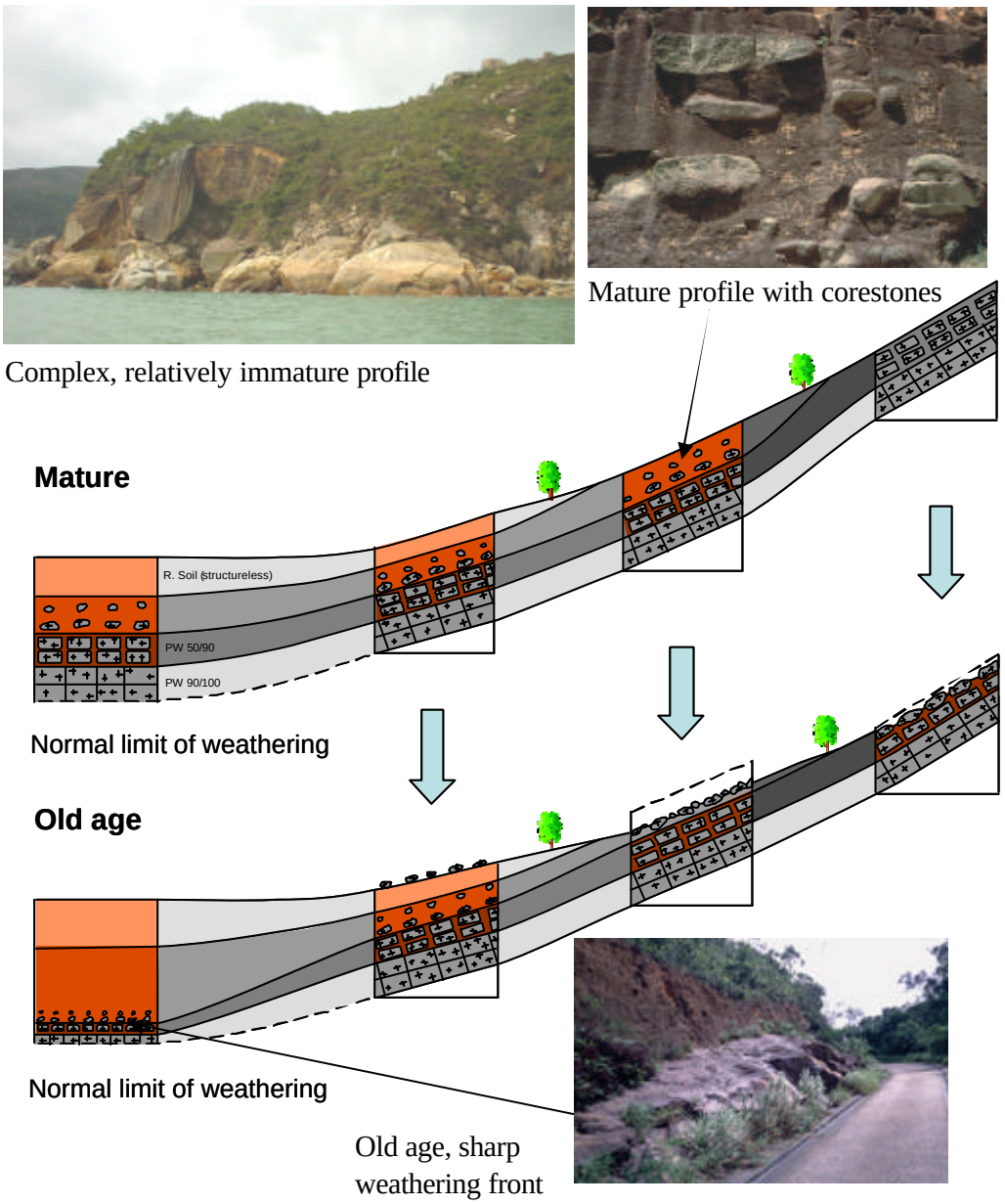


Figure 2 Examples of weathered terrain profiles (compiled from schematic diagrams presented by Ruxton and Berry, 1957)

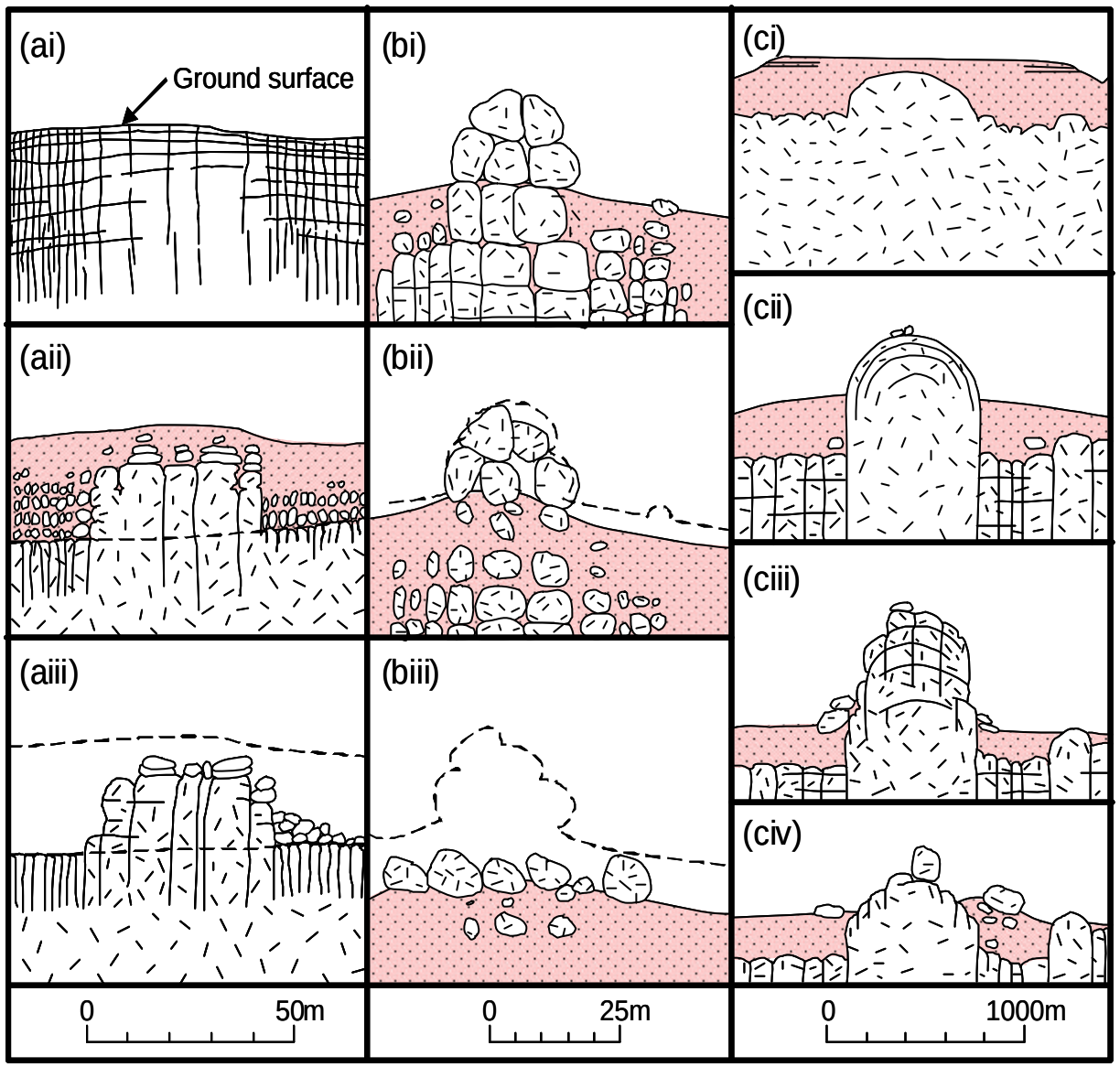


Figure 3 Other examples of weathering profiles. Redrawn from Selby (1993) ((a) after Linton, 1955; (b) & (c) after Thomas, 1965)

The examples of weathering profile presented by Ruxton and Berry (1957) and illustrated in Figure 2 are not exclusive. Other examples of profiles that might be encountered are illustrated in Figure 3.

In practice it can be extremely difficult to interpret the nature of the underlying profile simply from observations at the ground surface, either from air photograph interpretation or surface mapping. A large rock, partially exposed in a hillside, might be the upper surface of a tor as in (civ) in Figure 3; alternatively it could be part of an *in situ* boulder left behind as the weathering front proceeded at a more rapid rate beneath it as per (biii) above and as shown in the Ruxton and Berry example for shallowly dipping slopes at old age in Figure 2. Another possibility is that the rock is a displaced and partially buried boulder that has migrated from further uphill.

It is not always easy to tell which profile is the proper interpretation at a site without specific investigation and the wrong assumption will mean that the wrong geological model may be

adopted for design with possibly dire engineering consequences, as illustrated by Hencher and McNicholl (1995) with respect to a landslide in a cut slope on Tsing Yi Island (Choot, 1983a). Boreholes put down for the design of the slope had been stopped having proved 5m of rock (upper part of Figure 4). Additional boreholes put down after the failure indicated that the original boreholes had terminated in large corestones and not below rockhead as had been assumed for the design. The eventual slip surface passed beneath the corestones through what had wrongly been assumed to be rock.

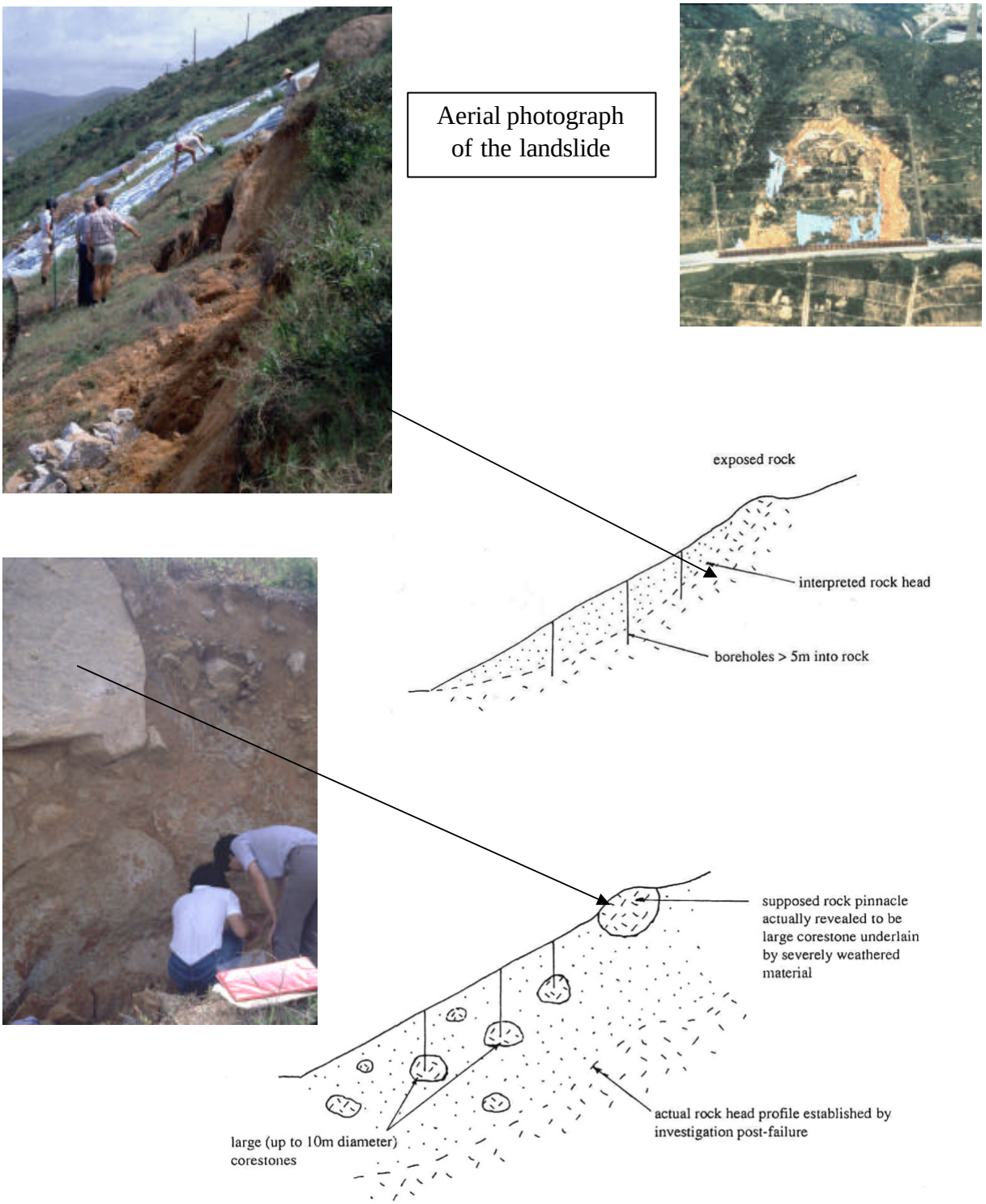


Figure 4 Expected and actual weathering profile – Tsing Yi landslide, 1982 (sections from Hencher and McNicholl, 1995)

Weathering Processes
The most important weathering processes active in weathered rock profiles in Hong Kong are the following:

- *Decomposition*: The result of chemical changes on exposure to the atmosphere (H_2O; CO_2; O_2). The original rock minerals, stable at the temperatures and pressures operative at the time of formation, break down at the Earth' s surface to clays and silts.
- *Disintegration*: Inter- and intra-grain crack growth and coalescence of cracks to form fissures and large scale joints
- *Eluviation*: The soft, disintegrated (or dissolved) material is washed out from the parent rock fabric through open joints or from the porous skeletal structure and deposited elsewhere (*illuviation*).

Engineering Consequences
Weathering processes lead to changes in mechanical characteristics. In particular, these are:

- At the *material* scale, which is the scale of core logging and laboratory testing of intact samples, there is growth of voids and change in mineralogy. These changes result in reduced density, loss of strength and stiffness and generally an increase in permeability. The rock turns to soil.
- At the *mass* scale, which is the scale of engineering geological models prepared for construction projects, the situation changes from one where a rock mechanics approach is clearly appropriate for investigation and analysis to one where the mass is made up of much weaker materials and soil-like. That said, for soil-like weathered rocks, relict structure, fabric and texture from the parent rock may be very important for engineering behaviour and performance even though the material strength is reduced to that of soil (broken down by hand). At intermediate stages, mixed hard and soft materials at the mass scale cause problems for piling, drilling and excavation (Shirlaw et al., 2000). At a basic level such heterogeneous mixed masses are difficult to characterize, test and derive parameters for and to analyze realistically.

Engineering consequences of weathering are dealt with in more detail in later sections.

Hydrothermal Alteration
Generally it is to be expected that the quality of the weathered rock mass will improve with depth although there may be exceptions with more weathered zones occurring beneath less weathered rock, as discussed earlier and illustrated in Figures 2 and 3.


Figure 5 China clay pit, Cornwall, UK

At some locations rock is found that has been decomposed by processes other than sub-aerial weathering. Such processes include the passage of high temperature groundwater or hot gases and fluids along faults or associated with the emplacement of igneous bodies. These rocks are termed hydrothermally altered. Hydrothermally altered rocks

are found with a similar range of strengths to grades of weathered rock but there may be other differences in engineering characteristics as discussed by Hencher et al. (1990). For example, grade V (as defined later), hydrothermally-altered granite can have a higher density than the equivalent grade of weathered rock because of its different origin. This may be due to a lower degree of eluviation in hydrothermal rocks so that the amount of residual clay that remains locked into the parent rock fabric is greater. Large masses of granite in UK have been hydrothermally decomposed, resulting in commercially exploitable deposits rich in kaolin (Figure 5), whereas equivalent tropically weathered granite in Hong Kong could not generally be mined commercially in that way.

In Hong Kong hydrothermally altered rocks are occasionally encountered, often associated with concentrations of kaolin and other minerals such as epidote and tourmaline. Such occurrences are even less predictable than the nature of weathered rock profiles. Figure 6 shows the anticipated ground conditions for the Black Hills tunnels (Mass Transit Railway, Tseun Kwan O extension). During tunnelling the majority of rock proved to be generally as anticipated; typical rock conditions are shown in Figure 7.

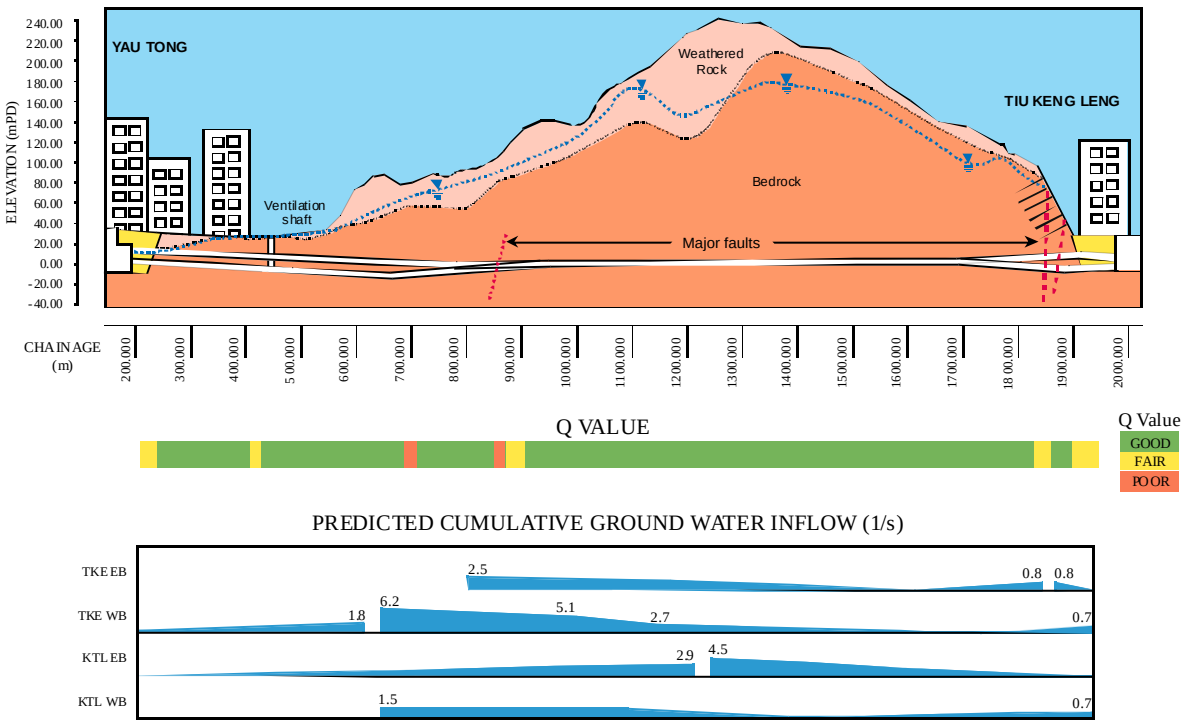


Figure 6 Black Hills tunnels, anticipated conditions

Figure 7 “Normal” rock conditions encountered in the Black Hills tunnels

In the event, with tunnel cover of perhaps 200m, hydrothermally altered granite was encountered unexpectedly in the main drive and could be traced as a zone across several of the roughly parallel tunnels (Figure 8). It caused a collapse and some delay. Figure 9 shows one of the tunnels with hydrothermally altered material exposed in the invert. The hydrothermal origin of the weak zone was indicated by its location at depth, with stronger rock overlying, and by its association with relatively rare assemblages of minerals.

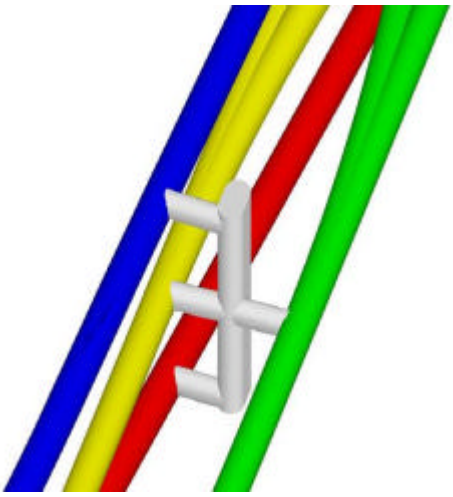


Figure 8 Crossover tunnels in the Black Hills project

Figure 9 Hydrothermally altered rock in the tunnel invert, Black Hills tunnel

DECOMPOSITION AND REDUCTION IN SHEAR STRENGTH

As weathering progresses the rock is weakened considerably. Generally it is convenient and practical to classify the process into six stages based essentially on strength variation (Anon, 1995). Others have tried to do this essentially scientifically using measurable chemical changes but such methods are not practical for field description and have not ever been correlated to engineering characteristics in any useful way to the author's knowledge.

Definitions of Weathering Grades

One thing needs to be made clear; weathering grades are not absolute but are a matter of definition. Grades are only useful for description if engineers and geologists use them as defined and do not reinvent them because of personal "experience" and preference. The six stages, which are based on the original work of Moye (1955), are illustrated in Figure 10. Simple index tests are recommended for separating the different grades to encourage consistent recognition and description. This is a very important objective because otherwise people get the wrong impression of what the rock is really like and incorrect information is communicated. The use of index tests is discussed in detail by Martin (1986). Figure 11 shows examples of grades of weathered rock in Hong Kong.

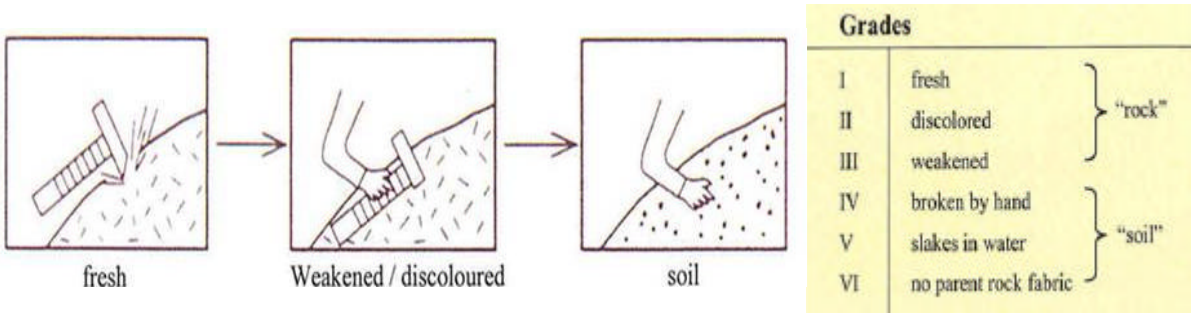


Figure 10 Grade classification for weathered intact rocks

Unfortunately, index testing is rarely conducted in practice, with reliance on visual impression to judge grades. Recent interviews with sewerage logging geologists in Hong Kong confirmed that none of them carried out index tests (Lam, 2005). Furthermore there is a tendency to describe all saprolites that can be broken down by hand as "completely decomposed" (Grade V) which is incorrect and no-doubt leads to over-design. In terms of stronger rock, people often incorrectly describe grade III as grade IV and therefore think that grade IV is stronger than it really is. It is a matter of misidentification and sloppy practice.

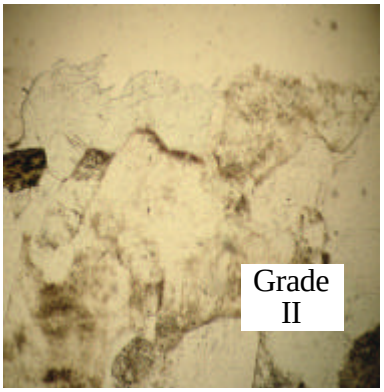
It must be noted however that, even if tests are conducted, there can be practical difficulties in assigning grades. For example a grade V (completely decomposed or weathered) granite, beneath the water table, will be saturated and placing a sample in water may not cause it to slake (disaggregate) rapidly. That does not mean that it is not at the grade V level of decomposition and weakness, just that the test does not work under those circumstances. Similarly some grade IV and even grade III rocks can be so fragmented that sufficiently large intact samples cannot be obtained to test whether or not they might have been broken by hand (Moye's original definition was with respect to NX core of diameter $2^1/_8$ inches.)

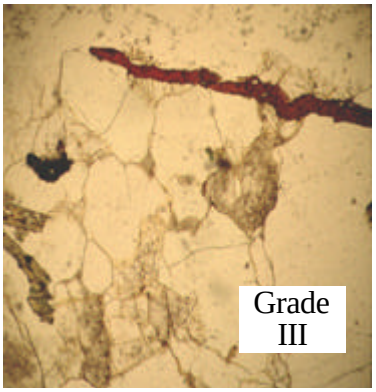
In the early 1980s a large research project was carried out in Hong Kong entitled, rather tortuously, "Cut Slopes in Hong Kong – Assessment of Stability by Empiricism" - allowing the acronym of CHASE (GCO, 1981). This study involved the collection of detailed

geological, hydrogeological and geomorphological data from more than 200, mostly very large, cut slopes, approximately half of which had had significant failures in the past. Data were collected as rigorously as possible and in a form that they could be examined statistically in an attempt to identify those parameters that were the most significant for stability (Brand and Hudson, 1982). For geological/geotechnical characterization, each slope was examined in detail in strips from crest to toe.

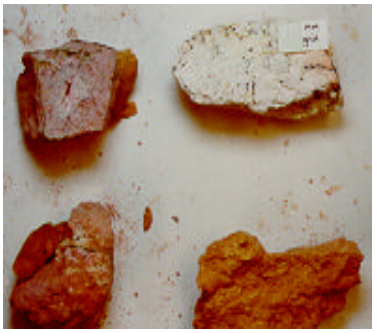
Because of the difficulties in defining weathering grades uniquely and the broadness of the grades, for the CHASE study, attempts were made to find other field indices that helped to define variations in degree of weathering (given that the Moye (1955) boundaries were to be taken as the basis for classification). It was also recognized that such tests, where gradational, might be of use in their own right in distinguishing rock at different stages of weathering.

To this end, time was spent examining many exposures, assigning Moye grades to samples as best they could be judged and then trying a range of other tests to explore the usefulness of each test in defining the degree of weathering. Of the various tests tried, the most useful repeatable test proved to be the Schmidt hammer (N & L types). It was thereby found that generally the point where material slaked completely (Moye's boundary between grades IV & V) seemed to equate to the point where there is essentially zero rebound from the hammer. That means *no* rebound at all despite repeated hammering at the same point and trimming away around the indentation to allow the test to be repeated many times. Obviously the test when used in that way on disturbed and probably densified material after repeated hammering (grades IV, V & VI especially) is very much an index test with the hammer simply providing a constant energy source and convenient rebound scale. No correlation would be possible with compressive strength from the standard charts (Brown, 1981). By trial and error it was similarly determined that the point where intact rock can be broken by hand, the boundary between Grades IV & III, equates approximately to a rebound value of about 25. This was for numerous tests on granitic and volcanic rock samples throughout Hong Kong.





Thin sections through grades II and III granite; normal light; grains about 2mm



Slake test on weathered volcanic samples

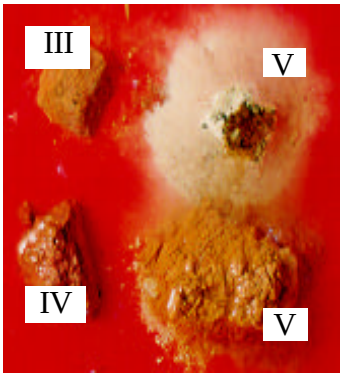


Figure 11 Examples of weathered grades

It is noted that other authors such as Irfan and Powell (1985a & b), Irfan (1996) have suggested grade correlations with other ranges of Schmidt hammer values (some overlapping) and these were included in Geoguide 3 (GCO, 1988). It is this author's opinion that such refined correlation is not practical or realistic given the imprecision of the original definition indices such as "breaking by hand" and that it just adds to confusion which de-motivates geotechnical engineers from carrying out index tests at all and therefore indirectly encourages misidentification. It also seems to imply that weathering grade is some kind of absolute condition that varies from rock type to rock type, whereas in fact the index type classification really relates to different strength states and conditions of deterioration. It is true that the same tests will not always be appropriate or useful for grading all rock types (see discussion in Anon, 1995) but, from the author's experience, the tests advocated here are generally useful for differentiating condition states of a wide variety of silicate rocks including igneous, metamorphic and some sedimentary rocks such as sandstones that in their strongest states are s*trong* or stronger.

Difficulties in assigning grades, especially to physically fragmented but not severely decomposed samples, became apparent during the CHASE study. For example more than 30% of samples that were ultimately classified as grade III had Schmidt hammer rebound values lower than 25 which is the general grade boundary (linked to breakage by hand). The reasons for such low values in grade III was generally the presence of open joints behind the face leading to "drumminess" and rebound values unrepresentative of the grade. It must again be emphasized that these tests are intended for grading intact material – not the rock mass. Of course the fracture state as reflected in low rebound values is significant in terms of rock mass quality but that is a different matter from weathering grade and should be dealt with elsewhere in the description.

It might be noted that decisions on grade classification can have serious contractual implications in Hong Kong despite the inherent difficulties in precise definition, particularly where the parent rock is closely fractured or disintegrated. For example the adequacy (capacity) of most bored piling in Hong Kong is assessed with reference to a table for allowable vertical bearing pressure for foundations on horizontal ground given in the Practice Note for Authorised Persons (PNAP) 141 produced by the Buildings Department (HKBD, 1995). One of the categories for allowable pressure in the PNAP table is *"1(b)"* that requires the pile to be founded on *"Slightly to moderately decomposed, moderately strong rock of material grade III or better…"* Difficulties arise where the rock is highly fragmented and the grade cannot therefore be adequately tested by any of the definition index tests. It is suspected that grade III rock is often wrongly described as grade IV which means that piles are taken deeper than contractually necessary.

Hencher (1986) suggested that a "basketful" of indices might be used when assigning grades and that leads to the opportunity for differentiating between say "loose" grade V and "dense" grade V and so on.

Internal Erosion, Deposition of Material and the Loss of Physical Bonding

Weathering is associated with the passage of groundwater through the vadose zone. The water introduces new chemicals that encourage decomposition of the minerals making up the country rock but it also acts to transport the decomposed and disintegrated debris out from the parent rock. Ruxton and Berry (1957) present a graphic example from Hong Kong. They describe how a tomb cut within saprolite and lined with loose brick walls was discovered at Li Cheng Uk. The tomb had been largely infilled with material washed out of the surrounding

saprolite and deposited within the tomb. They state:

"*…for every 2 tons of original Zone 1 debris, 1 ton is now external eluviated residue and the other ton was found illuviated into the tomb*".

In other words approximately 50% of the mass was internally eroded from the surrounding saprolite and transported into the tomb over a period of about 2000 years. This massive migration would obviously leave the remaining *in situ* material highly porous with a precarious structure with the potential to collapse. Figure 12 shows changes in granite porosity accompanying weathering measured by Ebuk (1991). Note the marked increase in porosity (intrusion volume of mercury) from Grade IV to Grade V. Such a reduction in mass and growth of voids is associated with loss of strength and increased compressibility.

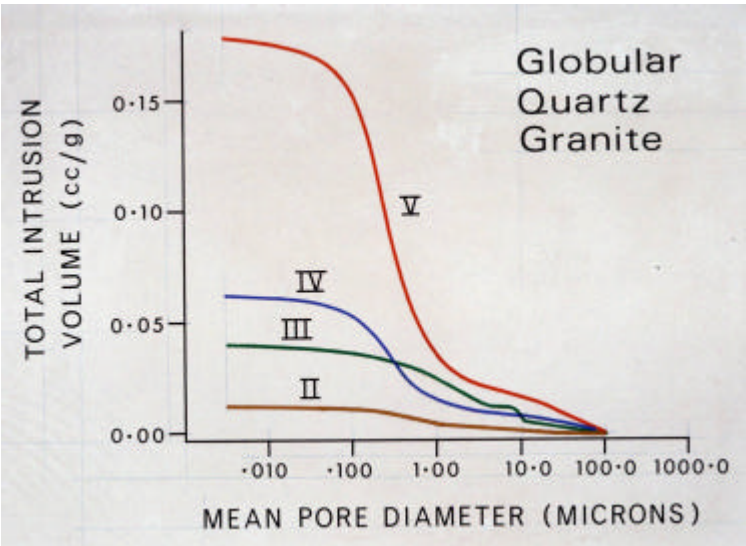


Figure 12 Growth of voids in granite (after Ebuk, 1991)

Anticipated Shear Strength

It is useful to consider the strength that weathered rock might be anticipated to have *in situ* prior to reviewing strengths that are measured in the laboratory.

Figure 13 is part of a rock strength chart for the Schmidt rebound hammer given in the ISRM Recommended Method (Brown, 1981). The range for grade IV material, 0 to 25, is given on the horizontal axis.

According to the ISRM chart, a rock with a rebound value of 25 and a density of 27 kN/m^3 would have a uniaxial compressive strength (s_c) of about 40 MPa. Rock at the grade IV / grade III boundary would however, be less dense so, if we assume a density of 20 kN/m^3, s_c of 28 MPa is derived for the top of grade IV from the ISRM chart. Figure 14 shows the Mohr circle for s_c = 28 MPa with a tangential strength envelope assuming ϕ = 38° which is about right. For this case cohesion of 7 MPa would be anticipated which is much higher than normally measured for strong grade IV material, as discussed later.

Considering the same boundary from another reference, the boundary between '*moderately*

weak" and "*moderately strong rock*" is defined as the strength where rock can be broken in two hands with difficulty (e.g. BSI, 1999) which is the same definition as that of Moy (1955) for the grade III/IV boundary. That boundary is stated to be the equivalent of a uniaxial compressive strength, s_c of 12.5 MPa. For rock with such a uniaxial compressive strength, the anticipated cohesion would be about 3 MPa which is still much higher than that normally measured in triaxial or direct shear testing. The reasons for this are discussed below.

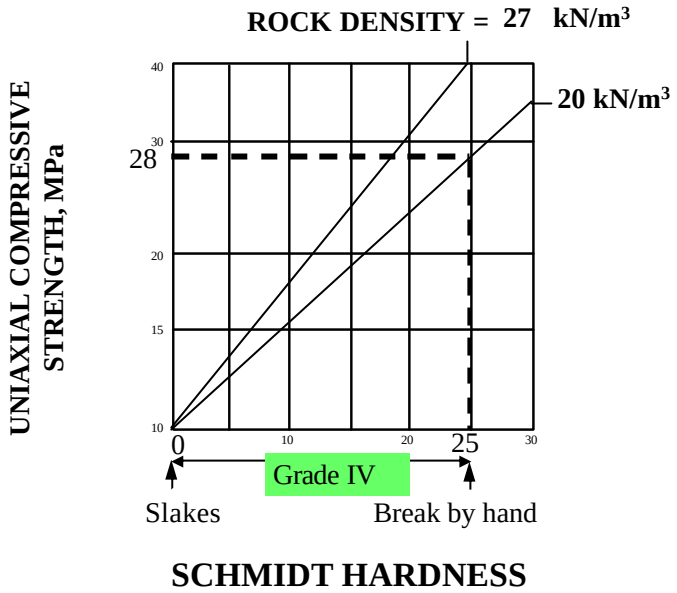


Figure 13 Compressive strength vs Schmidt hammer rebound number (Brown, 1981)

Measured Shear Strength

Generally shear strengths measured for weathered rocks are far, far lower than might be anticipated given their likely compressive strengths. Martin (1986, 2000) presents data from test on grades IV, V and VI decomposed granite and, whilst showing some variation in friction angle with grade, reports cohesion of less than 5 kPa for most grades (i.e. three orders of magnitude less than might be anticipated for the top of grade IV following the previous discussion). Similarly Geoguide 1 (GEO, 1993), which is commonly used as a reference for parameters in slope design, indicates cohesion of only 5 to 15 kPa for grade V granite.

Occasionally higher values are reported for samples with reliable weathering grade description. Figure 15 shows results from direct shear tests on samples of grade IV weathered granite taken from a trial pit within a failure scar on Tuen Mun Highway (Hencher, 1983b; Hencher and Martin, 1984). Whilst all the samples were classified as grade IV, the samples varied in strength and dry density. The crosses in Figure 15 represent measured peak strengths and show considerable scatter. Dilation was measured throughout tests and when "corrected" for that work the "non-dilational" basic friction angle (essentially the same as critical state) was well defined for all samples as about 38 degrees – the same as for many rock joints (Hencher and Richards, 1982, 1988). The angle of dilation and uncorrected peak strength increased with increasing dry density (and decreasing degree of weathering). One of the interesting things about these data is that *all* the variable strength is accounted for by dilational work as if the materials were sands of different densities. True cohesion was expressed as dilational work in testing and approached 300 kPa for the denser samples.

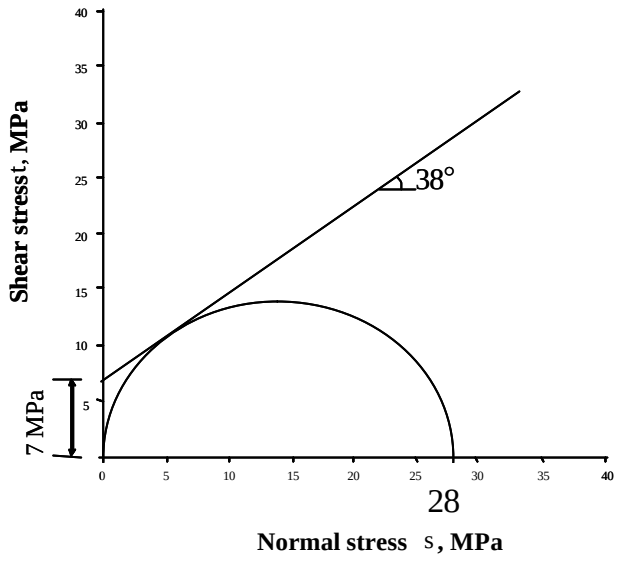


Figure 14 Anticipated shear strength at grade IV – III boundary on basis of Schmidt rebound value

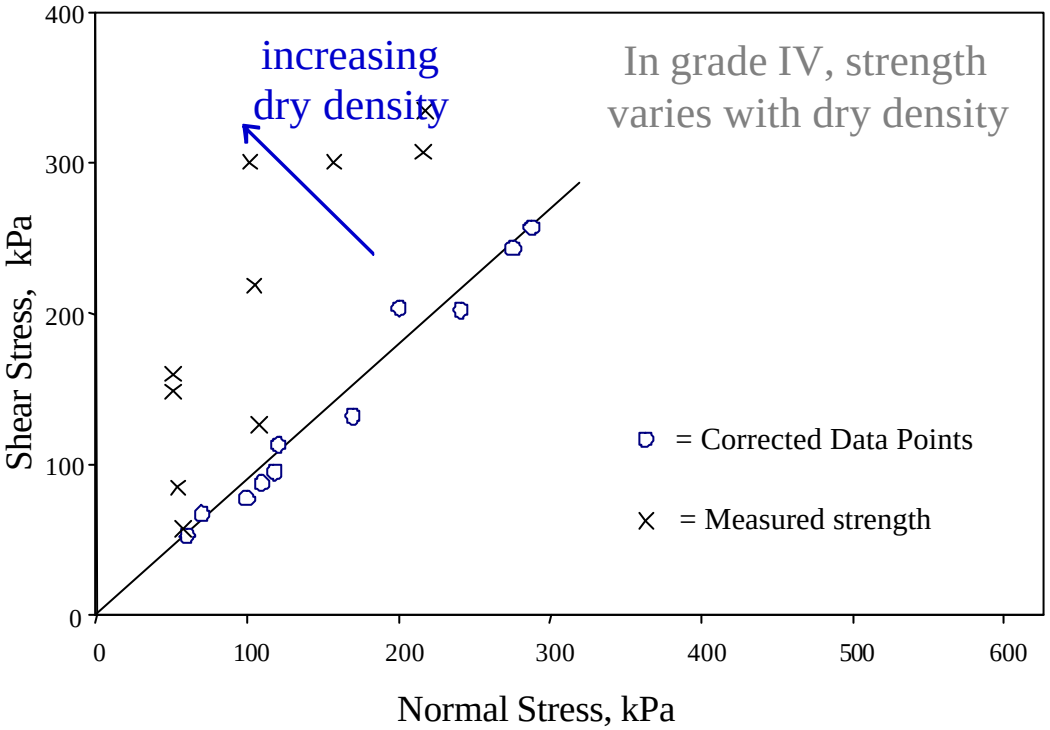


Figure 15 Direct shear test data from Tuen Mun Highway (after Hencher and Martin, 1984)

Disturbance
The main factors controlling strength of rocks are:

- Primary bonding between the different minerals. This is an adhesional strength.
- Diagenetic bonding – for example sand grains will deform and interpenetrate during burial with the growth of authigenic quartz at the junctions.
- Cementation– generally deposited from fluids or gases passing through the rock.
- Intact strength of mineral grains.

These factors contribute both to tensile strength and cohesion during shear. During shear, strength may also be derived from:

- Friction between grains.
- Dilation due to overriding which in turn depends on density and hence degree of interlocking and the condition of the mineral grains.
- Matrix infill (e.g. clay) within the skeletal fabric of relict grains.
- Suction forces which will increase the effective stress and thereby the frictional resistance.

Without doubt, one of the prime reasons for measured strengths in weathered rocks (grades IV and weaker) being lower than anticipated theoretically is disturbance during the sampling and testing process. Figure 16 is an attempt to illustrate this schematically.

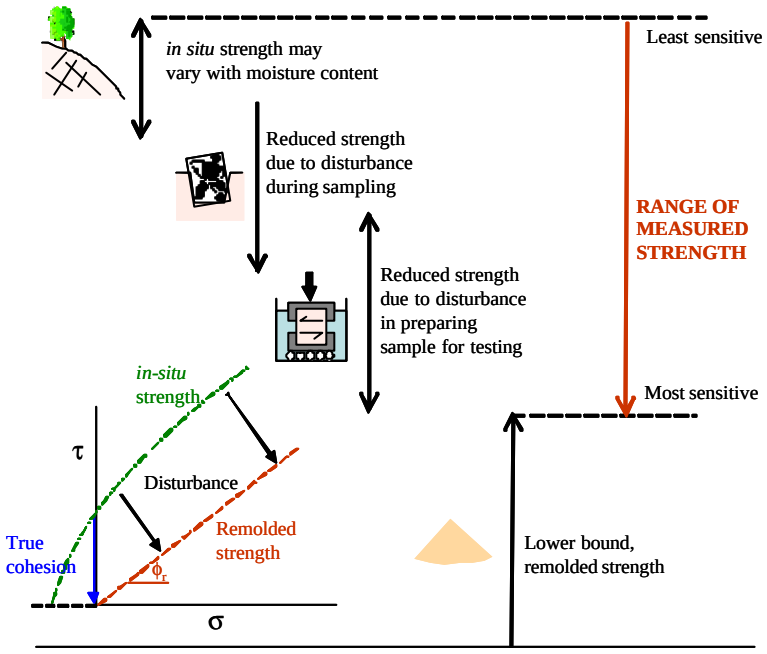


Figure 16 Sources of disturbance leading to reductions in measured strength

If the relict bonds are strong (for example in grade III rock), then the rock will be relatively insensitive to such disturbance and measured values in the laboratory may be close to the available strength *in situ.* A more sensitive, porous grade V/IV material may well be considerably disturbed despite great care in sampling and preparation. That means it is unlikely that the *in situ* strength of grades IV and V materials, in particular, will be well represented by laboratory test results.

Ebuk (1991) carried out research on the shear strength characteristics of grades IV, V and VI weathered granite samples from Nigeria, UK and Hong Kong. For his research he employed several newly developed pieces of equipment including a sampling device that could be used directly in a direct shear box. This box, known as the "Leeds shear box" (Ebuk et al. 1993) was devised to reduce one of the sources of disturbance, namely trimming of field samples to fit into a laboratory device. The sample can be cut into the box and sealed, in the field, as illustrated in Figure 17.

Figure 17 Leeds sampling box (a) for trimming in the field; (b) with top and bottom caps attached and ready to be transported to the laboratory. Central aluminium spacers split into two halves and can be removed prior to shearing, minimising further disturbance to the sample.

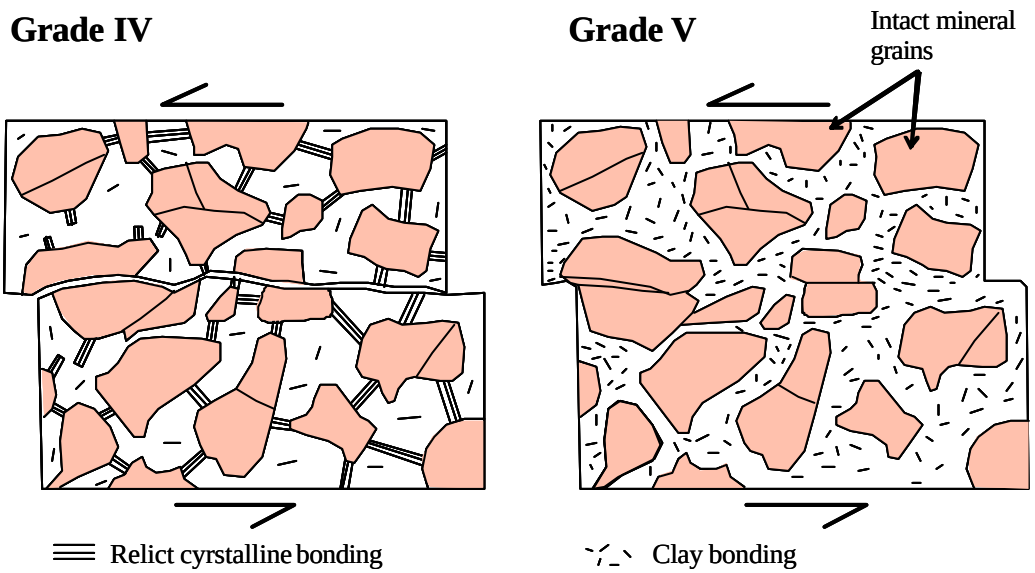


Figure 18 Idealised shear behaviour (after Ebuk, 1991)

Ebuk used this and other shear boxes together with an "analogue shear box" which was used to take samples through the same strain paths as samples in direct shear tests but that could be stopped at different stages so that the samples could be impregnated with resin and this sections prepared for microscopic examination.

One of his major observations was to confirm the decreasing role of relict bonding from grade IV to grade V as illustrated schematically in Figure 18. Differences in shear behaviour reflected these changes in state. For example, in Figure 19 it can be seen (topmost diagrams) that relatively strong grade IV exhibited brittle failure with a sharp and sudden drop in strength from peak. Moisture conditions had little influence on measured strength. Weaker material close to the grade IV/V boundary showed more plastic behaviour and reduced strength when saturated. Ebuk attributed this change in behaviour to softening of the clay matrix. Ebuk measured a range of strengths with cohesion up to 300 kPa for some grade IV samples (Figure 20) but this is still well below the anticipated 3 MPa or more for material of uniaxial compressive strength of 12.5 MPa as discussed earlier and so may still be far below the strength actually available *in situ* for such intact materials.

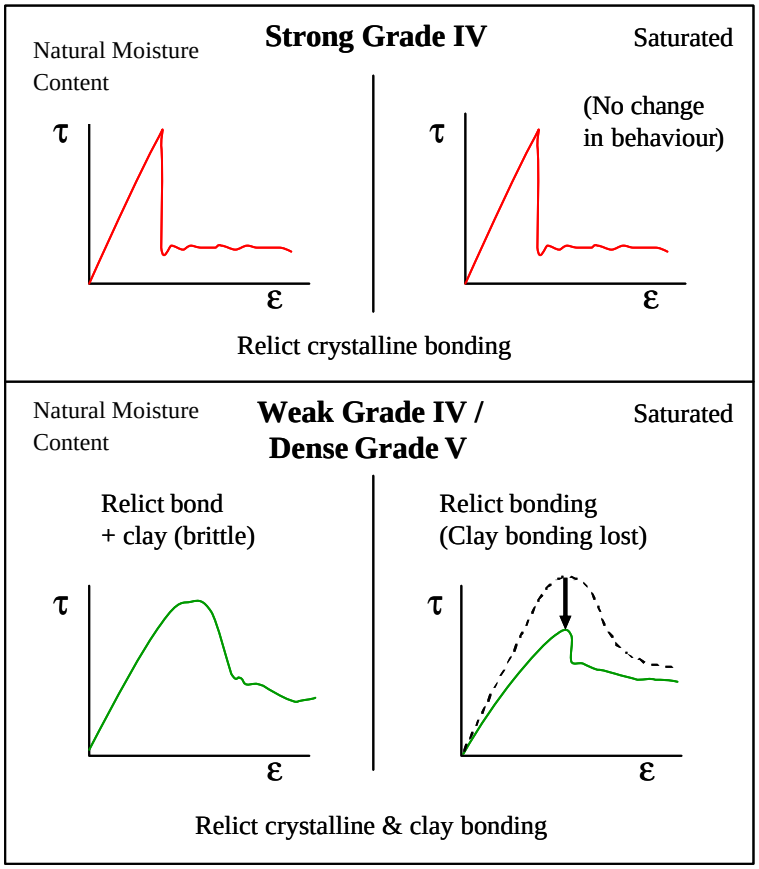


Figure 19 Influence of bonding type on shear behaviour (after Ebuk, 1991)

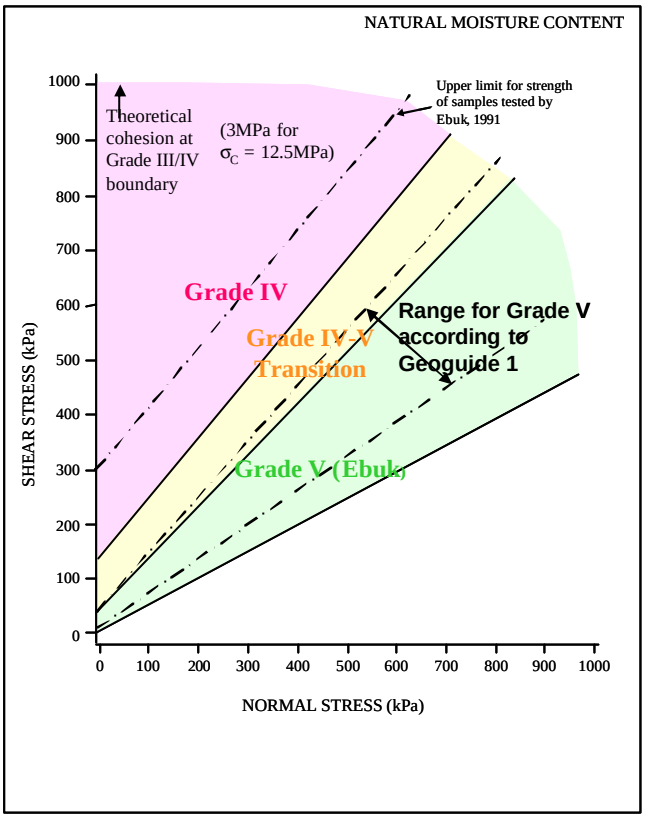


Figure 20 Influence of bonding type on shear behaviour (after Ebuk, 1991)

Disturbance During Engineering Works

It should be appreciated that weak, weathered rocks are not only susceptible to disturbance during sampling and testing but can also be sensitive to engineering works. Grade V material in particular, which is generally highly porous and notably slakes, can collapse and flow under adverse conditions (Shirlaw et al. 2000).

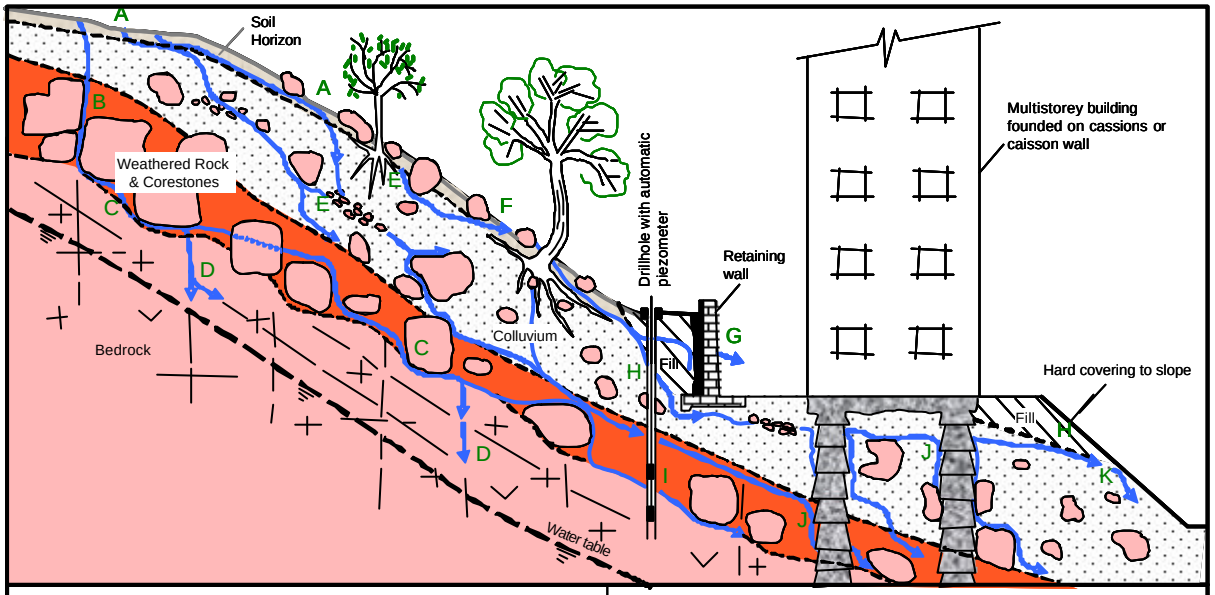
CHANGES IN PERMEABILITY

In general it might be expected that permeability and hydraulic conductivity will increase with the degree of weathering and that is the reported trend (e.g. GCO, 1982). This increase relates to the growth of pores and disintegration of the rock mass with extension of fractures leading to increased connectivity. However it is not quite so simple. At grades I to III, fracture flow will definitely dominate conductivity with localised channel flow being a very important factor (note the localised staining on joints in Figure 1). At later weathering stages the matrix rock (between discontinuities) will become more permeable so that flow is partly along joints and partly through the material. By grade V, the situation may be mostly matrix dominated but complicated by partial collapse and the redistribution of clays in some pores and discontinuities. By grade VI all the original rock fabric and structure is lost so the factors controlling permeability are completely different from those for rock masses comprising predominantly say grades I to IV.

One particular aspect of weathered mass permeability that is not well covered in standard texts on groundwater hydrology is the development of natural pipe systems although it is a commonly recognised hazard for the failure of dams, e.g. the failure of Teton Dam (Solava and Delatte, 2003).

Piping

As stated above, in rock, flow is generally along discrete channels on individual fractures (see for example Thomas and LaPointe, 1995) and it seems that such a channellised network often persists right though the various stages of weathering. There is now evidence of large scale underground streams in weathered profiles in Hong Kong and this type of dominant flow needs to be taken into account in investigation, hydrogeological modelling and design. Such natural pipes probably follow original structural paths (especially master joint / fault intersections), but may also be formed by seepage pressure in weak saprolite or in superficial soils such as colluvium. They also develop at permeability contrasts (e.g. colluvium overlying saprolite).



NATURAL SLOPE	DEVELOPEDSLOPE
A Groundwater inlet zones in small depressions, via cracks, animal burrows, root systems and around boulders.	G Pipe flow through poorly compacted fill. Retaining walls (with or without drainage filters) may cause damming.
B Movement of water along relict joints sometimes forming conduit flow.	H Pipe flow beneath fill and redirected by retaining structure.
C Movement of water around corestones, also influenced by relict joints and other 'permeability' boundaries.	I Piezometer locally intercepting pipe. Other pipes not intercepted.
D Fissure flow along joints both vertically and horizontally which may feed the more permanent water table.	J Voids caused by caisson construction redirect pipe flow.
E Pipe flow along gravel bands and around boulders	K Damming of pipe flow by hard cover to slope
F Efflux points on slope	

Figure 21 Schematic representation of piping in natural ground and close to developments (after Nash and Dale, 1984)

The development of pipes and their significance was recognised in the early 1980s in Hong Kong and a very useful summary paper was published by Nash and Dale (1984). One of their key figures, reproduced here as Figure 21, clearly indicates some of the potential problems that geotechnical engineers need to consider. Examples of failures associated with pipe flow are presented in Figures 22 and 23 from Hencher (1983b) and Choot (1983b) respectively. In fact, from the author's experience, a very high proportion of large landslides in Hong Kong are associated with pipe flow. Such pipes are often open (if active) but evidence of older pipe systems can be found in core samples where they are clogged with sediments ranging from well-sorted sands to graded bedding, indicative of underground stream systems depositing sediments into underground lakes, to gap-graded deposits.

Figure 22 Pipe in a landslide at Chai Wan Road, 1982

Figure 23 Pipe in Tsing Yi Pepco slope

That said, not all saprolites are characterised by the presence of pipe systems and, examples, in cut slope exposures are relatively few from the author's experience (see Figure 24). This observation which contrasts with the common occurrence of pipes in landslides, implies that the development of pipes might be linked to early stages of failure with the rock mass dilating and ground water flow exploiting the openings to erode out underground stream courses.

Figure 24 Pipes are seldom seen in undisturbed saprolite exposures

In 2003, evidence was discovered of a very large and unusual pipe system within weathered granite. A cavity, essentially a swallow hole, developed in a hillside above Yee King Road. Subsequent investigation showed that the cavity had chimneyed up from a large underground stream. Subsurface investigations and geophysical testing established the presence of several major, roughly parallel underground streams (Halcrow China Ltd, 2003). There was evidence that the underground streams were carrying sediment loads. Figure 25 shows layered deposits, including graded bedding, in a series of Halcrow buckets that had been installed down a borehole close to the cavity a few years prior to the incident. Evidently sediment-rich water had flowed down the borehole, past the buckets and escaped from the bottom into other, deeper stream courses (otherwise the borehole would have been choked with sediment, which it was not). The implied model is essentially that of transient turbidity currents flowing through a complex stream system following periods of high rainfall.

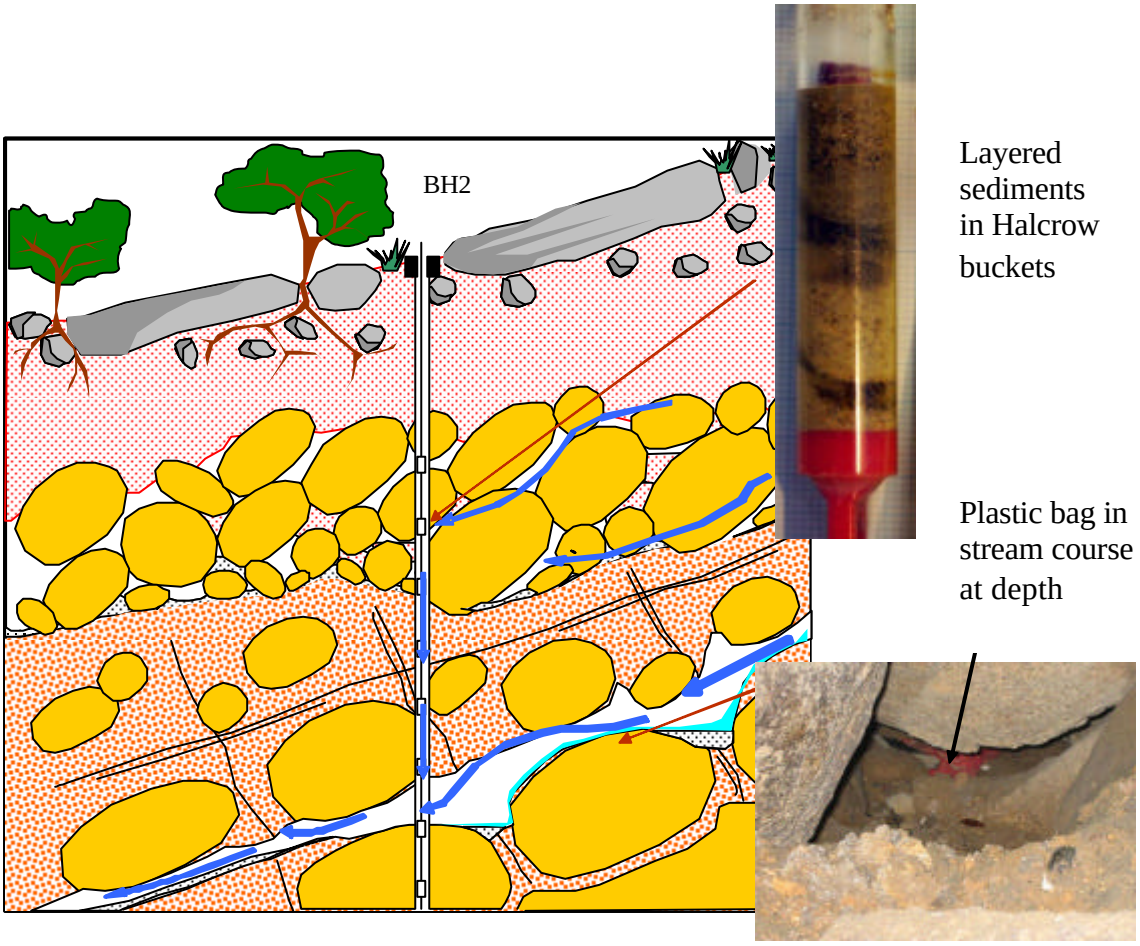


Figure 25 Evidence of underground streams close to the Yee King Road cavity

DISINTEGRATION

In Hong Kong the term completely decomposed granite is taken to be synonymous with completely weathered granite because that is the dominant nature of the weathering process which is in accord with the recommendations of Anon (1995). However a second common process during weathering is the growth of fractures at small and large scales, as emphasized by Selby (1993). In Hong Kong this is rarely acknowledged in description but it is an important process. Elsewhere in the world, especially in colder climates, the results of disintegration dominate the weathering product. Saprolite develops as a sandy soil of interlocking particles that are barely changed chemically. Such profiles are common in Korea (Lee and de Freitas, 1989) and other examples are shown from the USA in Figures 26 to 28.

Figure 26 Disintegrated granite – Nevada, USA

Figure 27 Detail of disintegrated granite detail from Nevada

Figure 28 Frost-shattering disintegration with core stone development – above Lake Tahoe, USA

Disintegration is also an important process in the development of weathered profiles in Hong Kong. During the CHASE study (GCO, 1981) discussed earlier it was found that disintegration sometimes made it difficult to assign weathering (decomposition) grades and required separate description. A three-class description method was adopted for degree of microfracturing – none, minor and extensive – with a special note where they were parallel (often parallel to the hillside and clearly relief features similar to sheeting joints). Examples are given in Figures 29 to 31.

Figure 29 General disintegration accompanying decomposition in Hong Kong

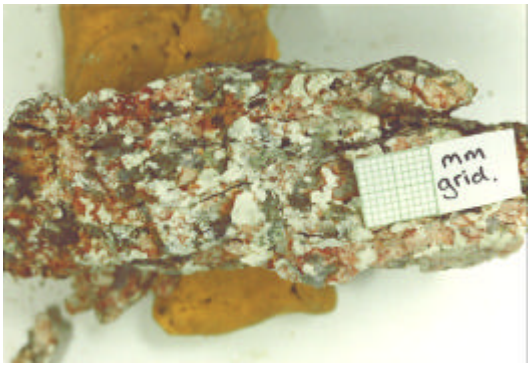


Figure 30 Parallel microfractures

Table 1 summarizes data from the CHASE study on microfractures for various grades of material in both granitic and volcanic rocks in Hong Kong. The grade IV granite exhibited most microfractures with 45% out of 132 samples described as having "extensive" microfracturing. By grade V less microfractures were noted which is indicative of healing from the grade IV to grade V state (largely because of the transformation of feldspar to clay and the redistribution of that clay into joints and cracks). The volcanic rocks generally showed less microfracturing, probably because of their different parent rock fabrics.

Figure 31 Extensive parallel microfractures in field

Table 1 CHASE data on microfractures

Microfractures									
		V		IV		III		II	
Granite	None	39	(130)	26	(132)	79	(51)	96	(53)
	Minor	35		28		18		2	
	Extensive	26		45		4		2	
Volcanics	None	91	(22)	82	(48)	88	(25)	100	(5)
	Minor	9		16		12			
	Extensive			2					

Notes:

- Numbers are percentages, sample nos. in brackets
- Microfractures will affect engineering properties and may affect judgment of Grade
- Joints are **distinct** from microfracturing
- Grade II/III granite has few microfractures whereas 45% of grade IV is extensively microfractured; Grade V has "healed" to some extent
- Volcanics are quite different to Granites with respect to observed microfractures

MACROFRACTURE GROWTH

It has long been clear to the author (Hencher, 1987), though not generally recognised, that many joint systems are only fully developed as visible and measurable structures on exposure to the elements. They may well have their origins as planes of weakness in some ancient tectonic or cooling event but they only fully develop as fractures during unloading and weathering. Selby (1993) agrees:

"the formation of joints is perhaps the most important single weathering process, even though it is seldom classified as such."

The concept that joints only fully develop during unloading and weathering is important because it then leads to the understanding that not all joints seen in the field as traces are actually fully-developed fractures. This is a particularly important concept when considering the fracture state (say by RQD assessment within the RMR or Q systems). When carrying out a line survey of discontinuities in a rock face or tunnel, all that is seen are traces, and there is little doubt that fracture frequency assessments based on trace measurements will underestimate the quality of rock relative, say, to an assessment based on intact vs non-intact core as per Deere's definition of RQD (Deere, 1968). In recognition of the importance of incipient fracture definition, Kirnig (1990) reviewed this subject as his MSc dissertation. Figure 32 is a classification based on his concept of a "life history of rock joints". At Class 1 the weakness is a preferred orientation of fracture, possibly discernible by microscopic examination. By Class 2 weakness directions can be seen as traces. This class also includes mineral veins that are bonded strongly to the country rock. This class will survive rough handling. Class 3 includes joints or other discontinuities such as cleavage and bedding planes that are incipient but relatively weak such that they can be opened up by poor drilling, mishandling or may develop on drying out of core. Classes 4, 5 and 6 are fully developed discontinuities. It is the author's view that only these three latter classes should be counted in RQD and Q assessments although it is recognised that Class 3 joints may be measured, especially where it is not certain that the joints have opened up due to handling/stress changes. Their inclusion in RQD and joint spacing calculations however may mean that the assessment of rock from borehole logging can err on the poor quality side relative to the in-situ condition. Engineers interpreting logs should be aware of this, especially in layered rocks, where the joint spacing plays a vital role, for example in the selection of tunnelling equipment.

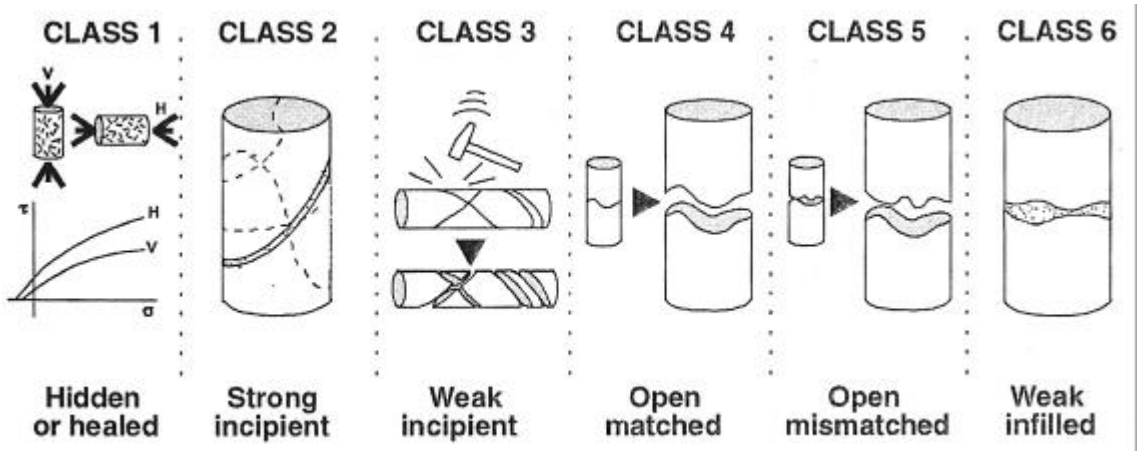


Figure 32 A "Life History" classification of rock joints

Evidence of active joint development is not well documented but there are some key examples. Martin (1994) reports site conditions at the Underground Research Laboratory (URL) in Canada. At the depth of the URL there are **no** joints in the granite but at shallower levels there are. Figure 33 shows stress relief fractures in the otherwise unjointed rock at several hundred metres depth within the granite pluton. Figure 34 shows stress relief fractures in core retrieved from depth. Interestingly the fractures in the rock close to the Earth's surface (where unloaded and more weathered) in some cases follow the same directions as appear as stress relief fractures in the otherwise unjointed granite (Martin, personal communication). Further details on this interesting and well researched site are presented in Everitt and Latjai (2004).

Figure 33 Stress relief in otherwise unjointed granite at great depth (Underground Research Laboratory level) (courtesy of Professor Derek Martin)

Figure 34 Discing on removal of core - URL Manitoba (courtesy of Professor Derek Martin)

Figure 35 shows three igneous contacts, at the ground surface, on Tsing Yi Island and illustrates that dyke contacts are not necessarily particularly weak planes even though they are clearly recognisable discontinuities (Class 2 in Figure 32). The author has heard it argued many times that dyke boundaries are weaknesses and expected locations for water flow but clearly that is an overgeneralization. Many other examples of strong, incipient discontinuities – faults, dyke boundaries and bedding planes - are illustrated by Fletcher (2004). Figure 36 shows a progressively developing fracture in a coastal granite exposure near Stanley.

Figure 35 Three rock types on Tsing Yi Island: note fused nature of interfaces

Figure 36 Fracture network under development at Stanley – come back in another 2000 years and what would you see?

The first occasion that the author realized that many fractures, including what, on first examination, appeared to be a parallel "tectonic" set of joints, only fully developed through weathering processes was when considering the stability of rock cuttings along Island Road, Hong Kong, when the road was to be widened in the early 1980s. Figure 37 shows what appears to be a set of tectonic joints through the volcanic rocks in a cutting. At one location it was noted that there was what seemed to be a sheeting joint passing through the supposed set of tectonic joints.

Figure 37 Island Road: a parallel set of "tectonic" joints on examination

That raised the question of why such an extensive through-going joint would develop when the stress causing that tensile fracture could have been relieved by movements along the existing "tectonic" joints?

The conclusion was that the tectonic joints could only have fully developed *after* the formation of the sheeting joint (which certainly appeared to be geomorphologically recent). That means that the parallel set of supposedly tectonic joints could only have developed as full joints after the sheeting joint and, geologically speaking, very recently.

Further evidence that many joints are only fully developed through weathering was provided by examination of core from the same site. Figure 38 shows an intact piece of core being set up for a direct shear test in a Golder Associates type shear box (Hencher, 1983). The surface to be sheared followed an incipient trace of one of the parallel joints (Class 2). Figure 39 show photographs and descriptions of the joint surface post shearing. Figures 40 and 41 show close-up views of the rock bridge that had kept the core intact. The coloured alteration bands surrounding the rock bridge are evidence of active weathering processes gradually eating away the bridge. Eventually that joint would have fully developed as a Class 4 joint with no tensile strength.

Figure 38 Island Road: incipient (Class 2) “tectonic” joint

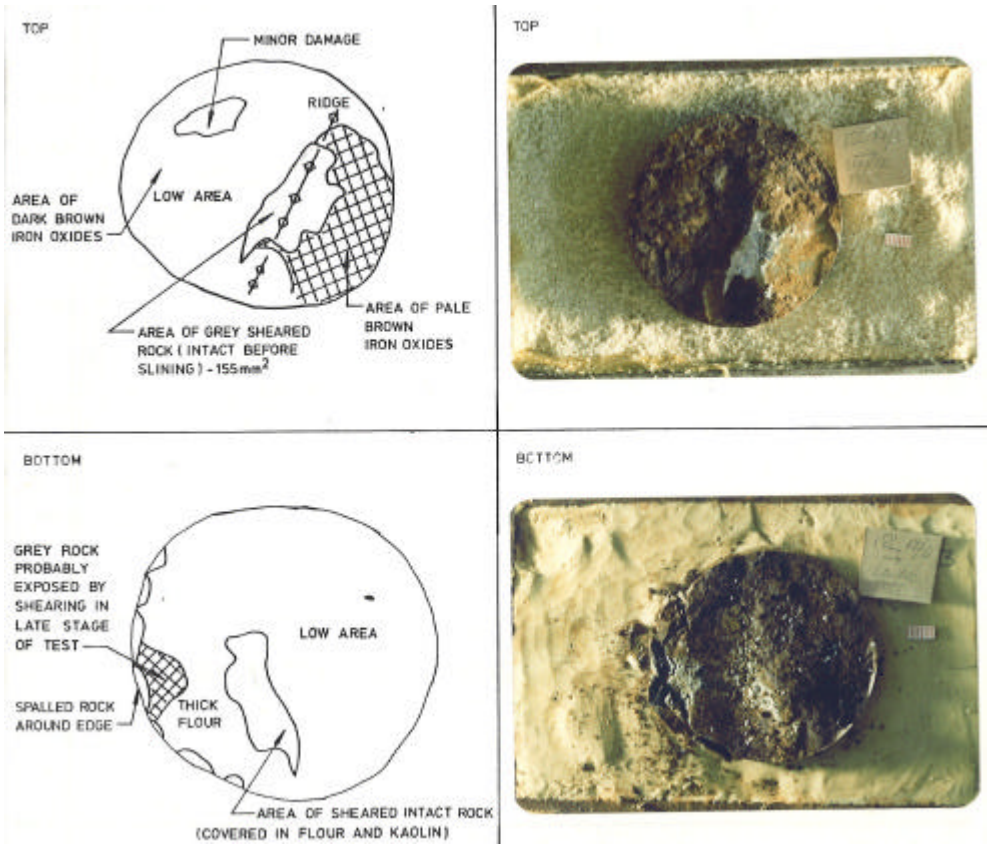


Figure 39 Rock bridge post-shear

Figure 40 Closer view of rock bridge

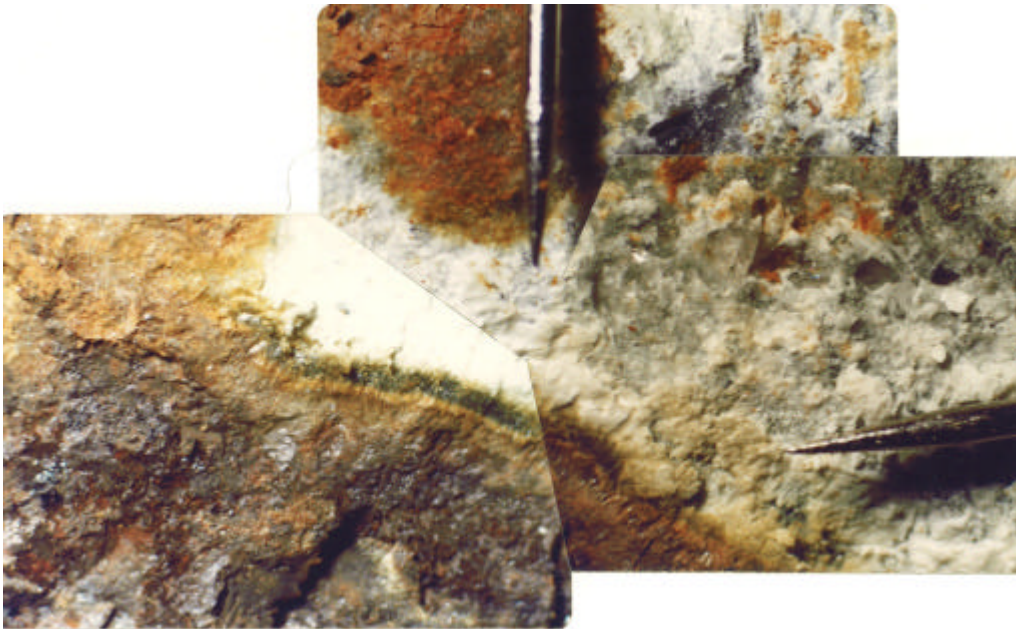


Figure 41 Island Road: rock bridge being slowly eaten away

WEATHERING AND CONSEQUENCES FOR THE SHEAR STRENGTH OF ROCK JOINTS

As noted earlier, the initial stages of weathering generally takes place along open discontinuities where water can permeate. The water and associated ions attack the exposed minerals and transport weathering products to new locations. Joint surfaces are therefore the loci for weak, weathered bands, erosion and smoothing of surface texture and the deposition of secondary minerals. All these processes can have marked consequences for shear strength. Examples of weathered products deposited along joints as coatings or infillings are given in Figures 42 to 44.

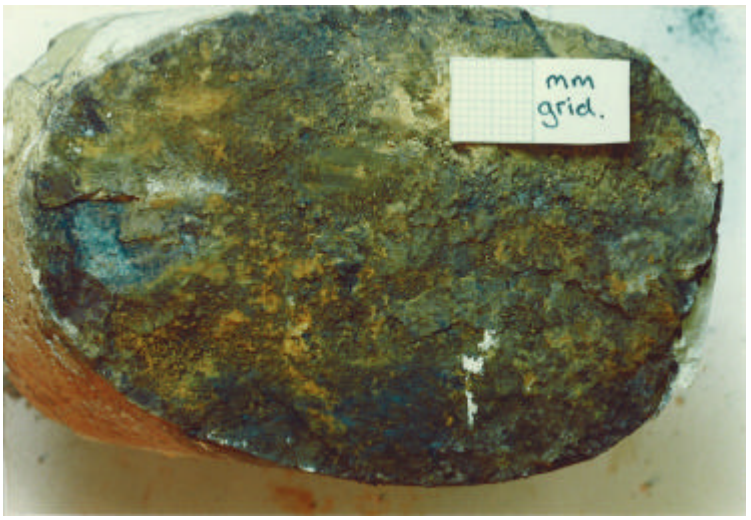


Figure 42 Iron oxide coated joint

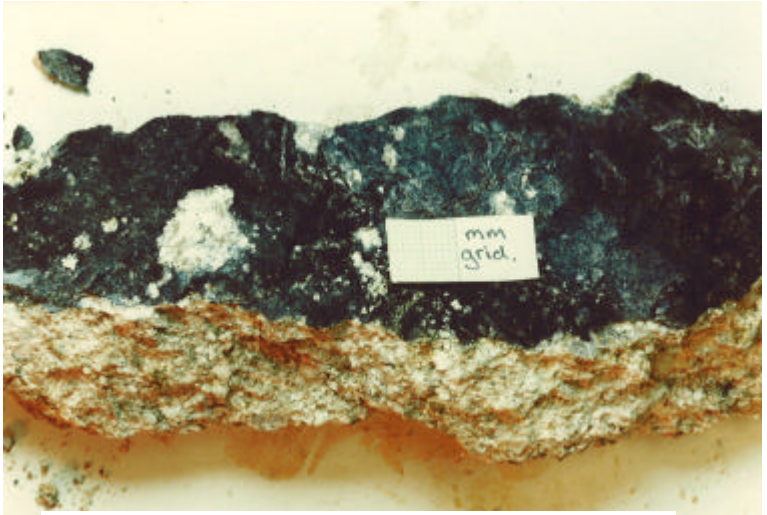


Figure 43 Manganese dioxide coated joint

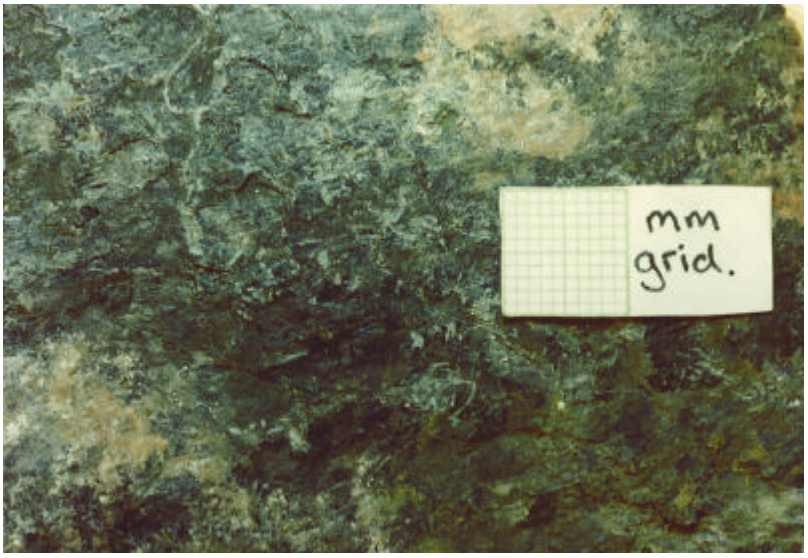


Figure 44 Chlorite coated joint

Yip Kan Street Rock Slope Failure

The Yip Kan Street failure of July 1981 (Hencher, 1981) illustrates the importance of weathering on the shear strength of rock joints. The failure took place on a dry day on daylighting joints that were in some areas only dipping at about 20 degrees (Figure 44). The parent rock is a coarse-grained quartz syenite.

Figure 44 Yip Kan Street rock slide

Figure 45 shows the results from a multistage shear test on an artificially formed tensile fracture through the slightly decomposed parent rock. This gives the upper bound for shear strength of an open, matched, class 4 discontinuity with no infill or coating.

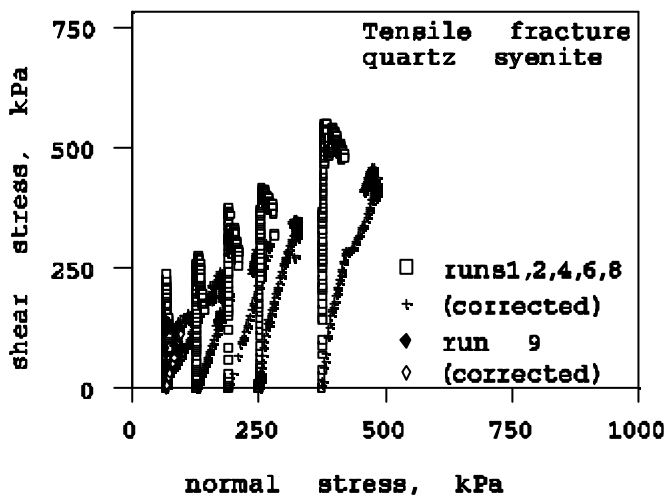


Figure 45 Artificial tensile fracture in grade II quartz-syenite (Yip Kan Street)

This experimental shear test was carried out to investigate the effect of repeated sliding on the roughness of the joint, peak shear strength and dilation-corrected friction.

The test was conducted under an initial normal stress (run 1) of 50 kPa, then, after cleaning off any debris, the joint was reset in its original position and sheared again under a higher normal stress (100 kPa - run 2). After run 2 and further cleaning off, the joint was reset and retested at 50 kPa. The processes was then repeated (i.e. at 150, 50, 200, 50, 250, 50, 350, 50 kPa). The 50 kPa runs following each of the increasing normal stress stages gave a measure of progressive damage to the joint. Essentially the test was simulating the decrease in surface roughness that might occur either through wear or weathering. Figure 45 shows selected test results both as measured and corrected for dilation (Hencher and Richards, 1988; Hencher, 1995). It can be seen that the corrected data define a basic non-dilational friction envelope (peak) quite well.

Figure 46 shows some of the data in more detail. It can be seen that the uncorrected data from runs 1 (a fresh, matching tensile fracture) and 9 (after repeated shearing), at the same normal stress, are very different because of the loss of roughness and surface texture by run 9. It is also noted however that the *non-dilational*, corrected data for peak strength from runs 1 and 9 is almost the same. This illustrates that loss of roughness will reduce peak strength considerably because of reduced dilation but the underlying non-dilational friction for natural joints is essentially a constant. Generally that friction angle is about 40 degrees for most silicate rocks (Papaliangas et al. 1995). Clearly the rock slide at Yip Kan Street could not have occurred on unweathered joint surfaces.

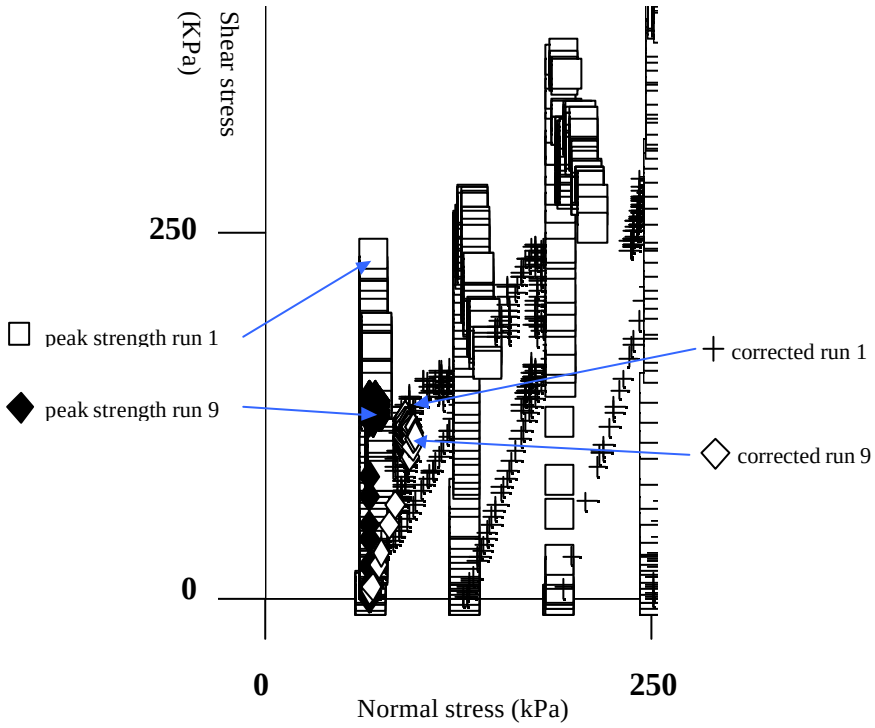


Figure 46 Details of tests on grade II quartz syenite

To help set limits, tests were also carried out on flat saw cut surfaces of the quartz syenite and these provided a friction angle of 28 degrees which is still higher than the angle of dip of the plane along which failure took place.

In fact the failure occurred along persistent joints coated with weathering products. There were two main coatings observed – iron oxide and chlorite. Direct shear tests were conducted on samples of both types of joint. The data are presented in Figure 47. It can be seen that the iron oxide coated joints exhibited non-dilational friction of about 38 degrees (up to normal stresses of 6MPa) which is consistent with values measured on similar coated joints in granite (Hencher and Richards, 1982). The shear strengths measured for the chlorite-coated joints however were much lower at the low stresses relevant to the failure and indeed were lower than for saw cut surfaces. The failure was thereby explained as due to sliding on chlorite-coated joints and highlights the importance of weathering processes on shear strength.

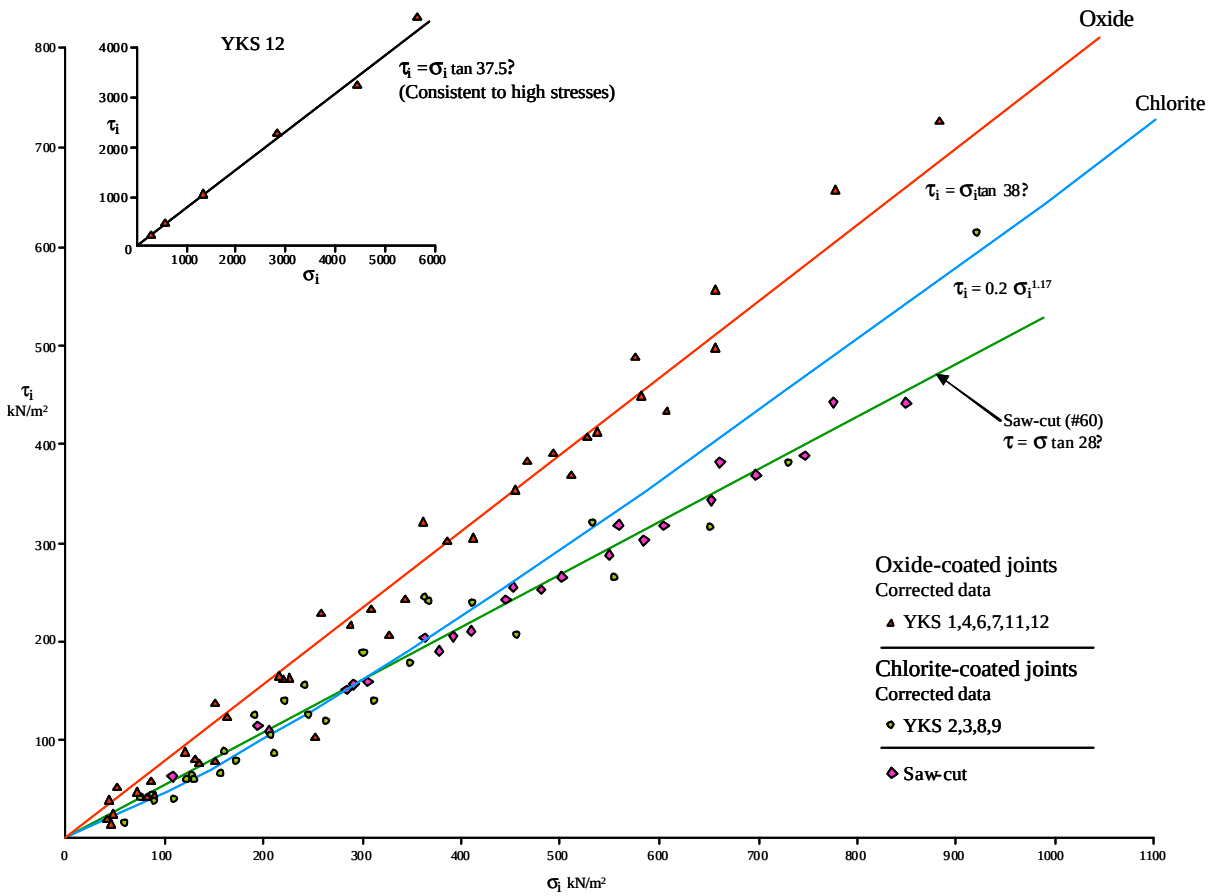


Figure 47 Direct shear test data (corrected for dilation) at Yip Kan Street

SLOPE DETERIORATION AND PROGRESSIVE FAILURE

Over recent years evidence for progressive deterioration of slopes prior to final detachment has become better documented in Hong Kong. This applies to both natural slopes and cut slopes (Malone, 1998; Wong and Ho, 2001; Parry et al. 2000).

It is obvious that slopes progressively deteriorate with time until eventually they fail. Ho (2004) reports that natural terrain landslides occur in HK at an average rate of 1 per 2 km^2 per year. Some of the factors influencing natural terrain failures (and their consequences) are illustrated in Figure 48.

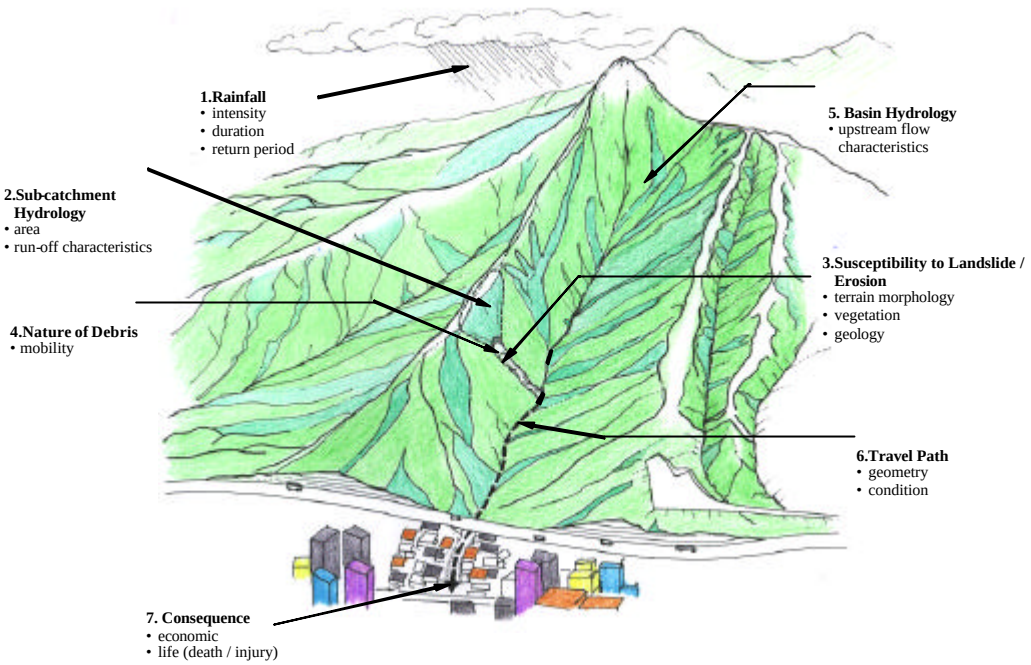


Figure 48 Factors contributing to risk from natural terrain landslides

The concept of deterioration as it relates to general geomorphological development is illustrated schematically in Figure 49. The gradual deterioration can be represented by a curve in which the Factor of Safety reduces over a period of time which may comprise tens or hundreds of years. The vertical lines represent temporary reductions in Factor of Safety caused by relatively short-term, transient events (days). In the course of time, the slope will deteriorate to the point where it is vulnerable to a transient event – causing a reduction in the Factor of Safety below 1.0. Whether that event results in catastrophic failure or only minor movement and internal distress depends on many factors including the severity of the triggering event and how long it lasts.

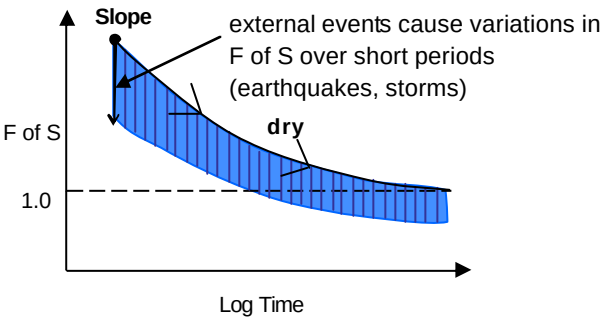


Figure 49 Reduction of Factor of Safety with time (long term and short term)

In general it can be surmised that for any given hillside, at any particular time, different locations will have different Factors of Safety. One might consider the hillside to comprise an inventory of different slopes of different susceptibilities. The susceptibility at each location will be a function of the weakness of the rock/soil at that location but also many other factors such as local slope angle (and therefore shear stress), catchment leading to that location (which will influence water pressures and erosion potential), local topography leading to concentrations of surface flow, erosion and undermining and vegetation cover (e.g. deep-rooted trees will help hold the soil together). In general however particular zones of the hillside will have different susceptibilities. In Figure 50 the concept of "old" and "new" terrains (Hansen, 1984) is illustrated schematically in terms of slope development. Location A is within the old terrain at the top of a hill, which is being progressively eroded at an active "landslide front" at Location B. Location C is in shallower terrain where the landslide front has already passed through. At Location D coastal erosion is active. The differences in susceptibility at each of these locations are represented in Figure 51. Away from the coast, location B has the highest susceptibility and rapid deterioration and even detachment failure can be expected in high rainfall events or earthquakes.

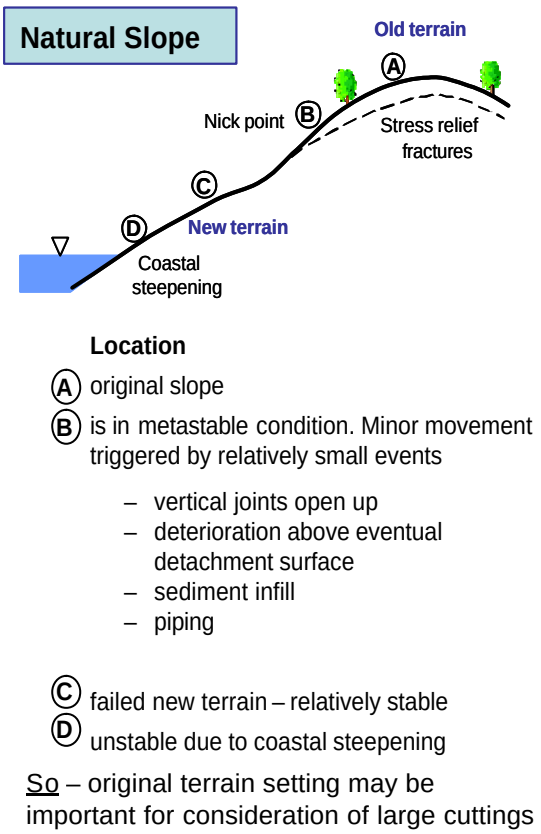


Figure 50 Schematic cross section through hillside

At B a minor rainstorm (say a one in 1 year storm) will probably not result in any discrete landslides although it will contribute to the general deterioration of the hillside which might be measurable given sophisticated instruments.

A more severe event (say a one in 10 year storm) may cause a few failures in areas where the ripened factor of safety is approaching 1.0 (say 1.0 to 1.1).

A much more severe event (say one in 100 year storm) may cause all slopes to fail within a much wider range (say 1.0 to 1.3). Not only will the intensity of such a storm initiate discrete failures but the duration of the heavy rain will make the debris more mobile so that it can flow a long way and impact on more structures than would otherwise be the case.

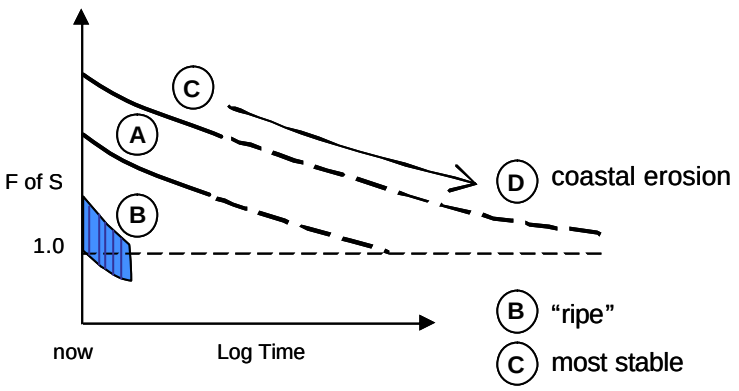


Figure 51 Susceptibilities at different locations on hillside

Cutting slopes will make them more susceptible to triggering events as illustrated in Figure 52. At the location where the cutting is made, the natural slope would have gradually deteriorated in geological time perhaps over hundreds of years. However the process of cutting the slope leads to a reduction in the Factor of Safety because of increased shear stress (over-steepening), reduction in confining stress and possibly more aggressive groundwater conditions.

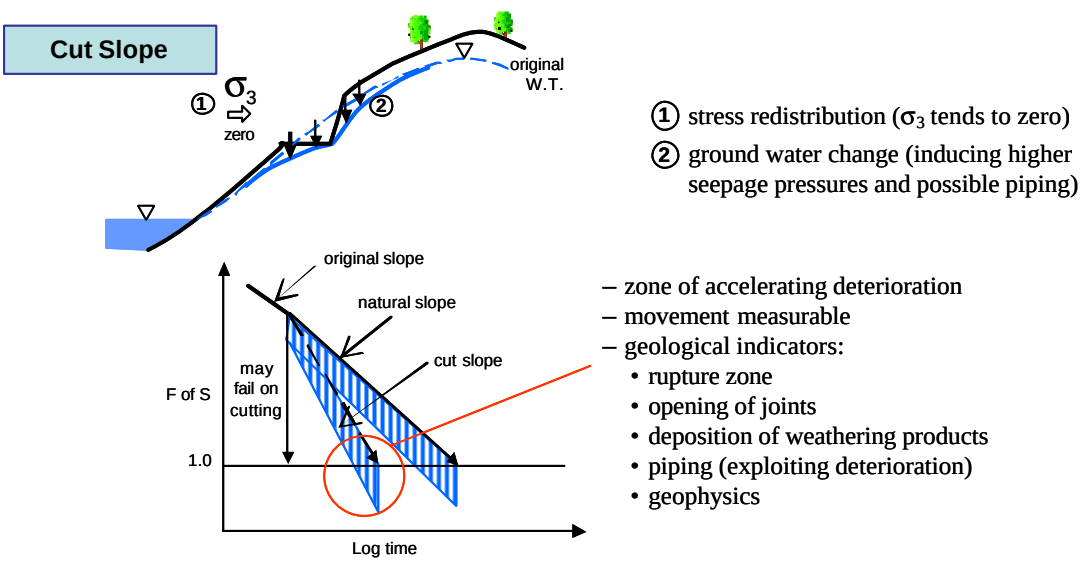


Figure 52 General concept of stability for cut slopes : geomorphological setting and deterioration

Signs of gradual deterioration or, more likely, the cumulative effect of intermittent triggering events can be seen in many exposures. Figure 53 shows a joint that is apparently developing with the formation of en-echelon fractures. Eventually it will fully develop as a through going fracture such that the overlying block will detach. Note the evident dilation (opening up) in the lower part of the joint.

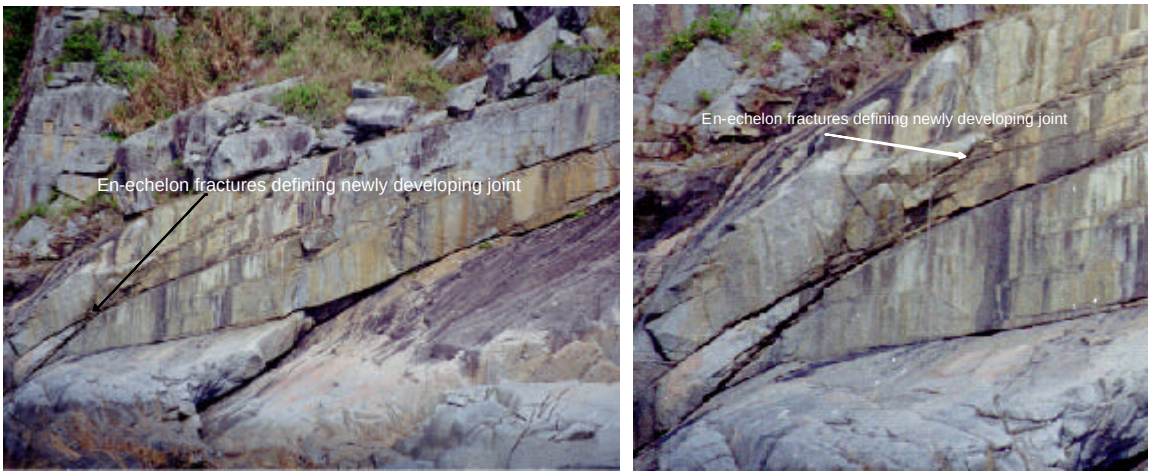


Figure 53 Fracture development in granite, Hong Kong

A Case of Progressive Failure prior to Detachment

Figure 54 shows the source area for a landslide in natural terrain that showed clear evidence for progressive deterioration prior to final failure (Halcrow China Ltd, 2001).

Figure 54 Source area for the Leung King natural slope landslide, 2000

The failure was essentially a translational rock slide on an undulating sheeting joint. In Figure 54 a central exposed area can be seen that is pale grey, almost unjointed granite. That part of the hillside (and below) has not moved. In contrast, the rock above is brown stained and contains many joints. That is because it has moved, resulting in joint development and extension because of stress concentration, and stained because of weathering processes.

Figure 55 shows the contact between the upper, brown and fractured, displaced rock and the underlying grey, unfractured granite.

Figures 56 to 58 show one of the waves in the sheeting joint with evidence of previous movement followed by infilling of the opened fractures with sediment. The landslide zone probably underwent repeated events during which the FoS was reduced temporarily below 1.0 with a cycle of displacement and weakening on each occasion. The process is illustrated schematically in Figure 59.

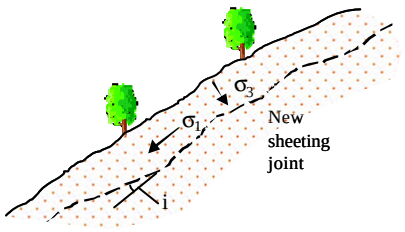
Figure 55 Fractured rock (partially displaced) above unfractured basal release surface (sheeting joint)

Figure 56 Wavy sheeting joint surface with measuring grid. The overlying rock slab has been displaced slightly downhill

Figure 57 Close up of part of Figure 56

Figure 58 Even closer view showing displaced granite blocks and clay infill

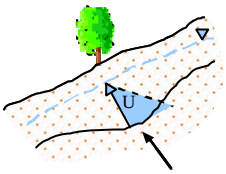
1.



- Rough, tensile fracture develops parallel to hillside
- Shear resistance = $s_3 \tan(f + i)$

  $f \approx 38°$ typically

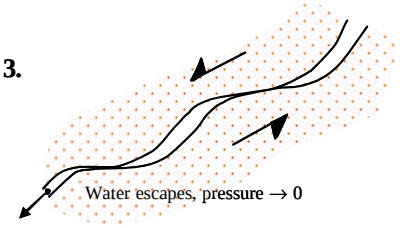
2.



- Water pressure develops in fracture
- Shear resistance reduces to

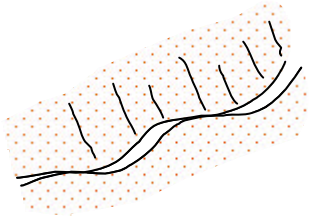
  $(s_3 - u)\tan(f + i)$
- Movement initiated

3.



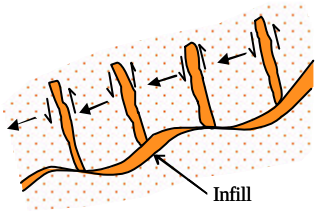
- Movement opens joint
- Permeability increases
- Water pressure drops
- Movement ceases

4.



- Tensile fractures develop in overlying displaced slab, now balanced on a few asperities.

5.



- Some continued creep movement (intermittent)
- Tensile fractures and sheet joint apertures become infilled with sediment
- Local vertical shearing on joints
- Note that rock (albeit weathered and weakened) still controls friction until waviness finally overcome

Figure 59 Deterioration of slope with intermittent movements on sheeting joint

Clay Infill as an Indicator of Displacement and Future Detachment

There are many case examples of dilation and fracturing of the rock mass, often associated with deposition of sedimentary clay, mainly kaolin, prior to final detachment. One example is the landslide at Junk Bay Road (Hencher, 1983c). The failure took place on a sheeting joint, patchily infilled with thick kaolin (Figures 60 and 61). Almost certainly, prior movement had taken place with dilation and deposition of kaolin in the hollows on the sheeting joint surface. An important corollary of this concept is that the joint would maintain rock-rock contact on waves prior to final detachment. In other words the clay infill, though important as an indicator of distress and probably affecting permeability, would not affect the frictional shear strength of the joint up to the final stage of detachment.

Figure 60 Junk Bay Road landslide

Figure 61 Kaolin-infilled sheeting joint at Junk Bay Road landslide

Another case is illustrated from a failure on Tsing Yi Island (Choot, 1983b) where there was evidence of previous movement (clay-infilled joints). Indeed the eventual "failure" involved the same mechanism as that illustrated in Figure 59 with limited movement involving dilation and the release of pore pressures allowing a temporary restoration of a FoS greater than 1.0. Figure 62 shows the slope with considerable signs of distress but not full detachment. Figure 63 shows an exposed joint in the rear of the failure, infilled with kaolin and slickensided with manganese dioxide. Almost certainly this slickensiding largely pre-dated the movement shown in Figure 62. Figure 64 shows a sample taken from the joint with thick kaolin infill, merging with the decomposed granite fabric and almost certainly having accumulated in an intermittently opening joint over a long period. Similar, more recent examples are discussed by Kirk et al. (1997) and illustrated in Fyfe et al. (2000). In particular the presence of clay-infilled vertical fractures as seen in the rear of the fatal Shum Wan Road (see Fyfe et al. op cit) and Fei Tsui Road landslides (personal observation) are possibly indicators of precursory movements in the author's opinion.

Figure 62 Tsing Yi, distressed slope, 1982

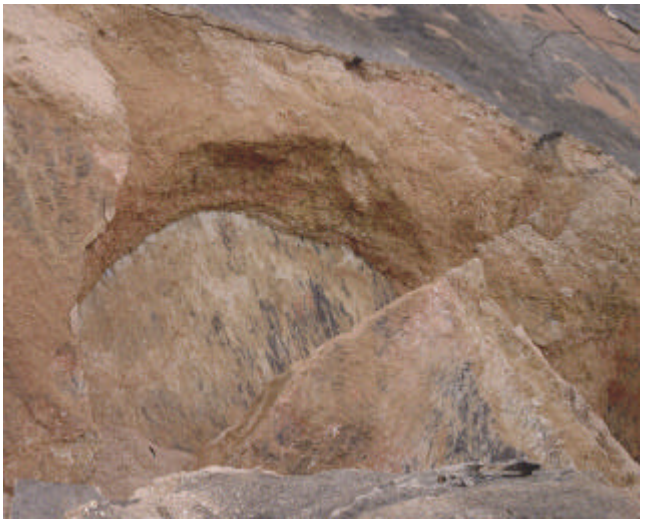


Figure 63 Tsing Yi : slickensided joint in back scarp

Figure 64 Sample from kaolin infilled and slickensided joint

Evidence of Prior Movement at the Ching Cheung Road landslide, 1997

The Ching Cheung Road landslide in 1997 was a serious event. It blocked a major road and was notable in that the slope had been upgraded a few years previously (albeit without any preventive works being carried out in the area that failed). The case is reported in Halcrow Asia Partnership (1998). The failure was the latest of a series of failures that had taken place along that stretch of road over 20 years or so since road construction (Hencher, 1983; Hudson and Hencher, 1985). Figure 65 is a digital terrain model of the area of the 1997 failure which occurred in Slope No. 11NW-A/C55 and Figure 66 is an aerial photograph taken in 1995 before the failure. Figure 67 shows the slope, post failure, with some remedial shotcreting works. Figure 68 is a cross section showing some of the geological features contributing to the failure, including the presence of persistent, adversely-dipping decomposed dolerite dykes. In essence however, the failure was simply a case of an over-steepened slope in weak decomposed granite affected by high water pressures.

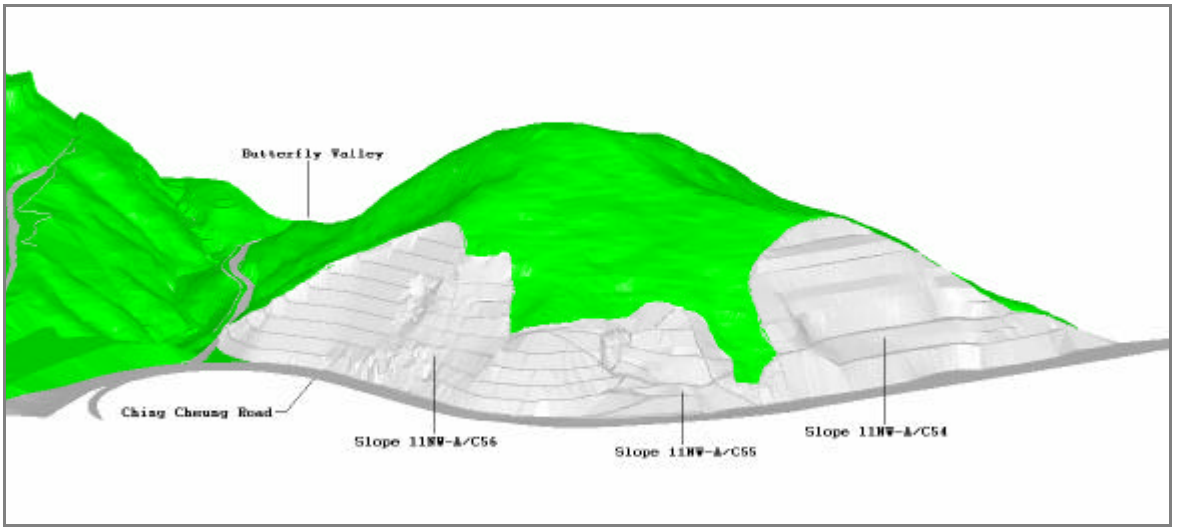


Figure 65 Digital terrain model of Ching Cheung Road

Figure 66 Ching Cheung Road, 1995

Figure 67 Ching Cheung Road, Aug 1997

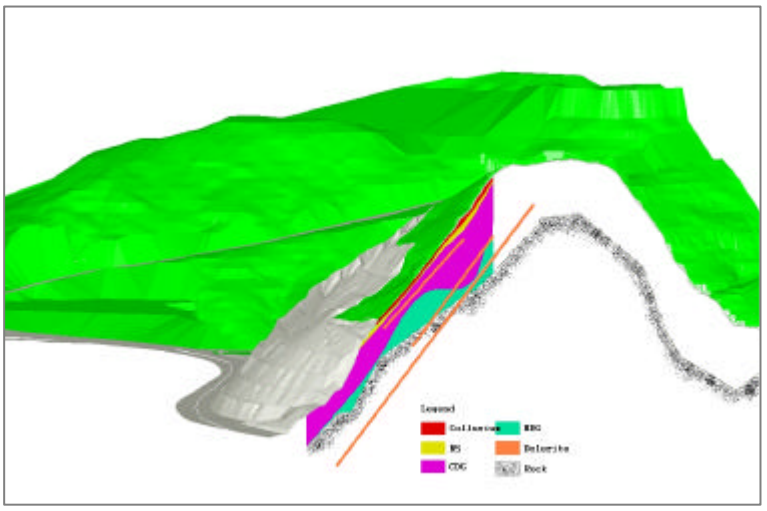


Figure 68 Geological cross section – Ching Cheung Road landslide

In the context of this paper the interesting aspects are the precursory signs of failure. Desk study for an adjacent failure in 1983 (Hencher, op cit) had identified an old failure at the location of the 1997 failure. Several of the photos showed previous distress; an example is given in Figure 69.

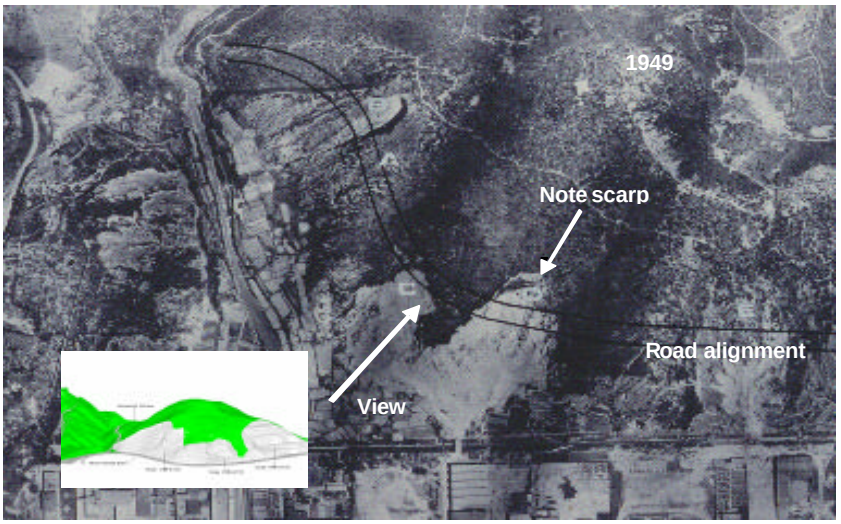


Figure 69 Aerial photograph taken in 1949. The road alignment is marked and the scarp at the location of the 1997 landslide highlighted.

A detailed investigation was carried out after the 1997 failure including numerous boreholes with continuous Mazier sampling and deep trial pits. Abundant evidence was found of precursory distress and associated geological features. One of the most interesting was the presence of numerous sediment-choked natural pipes in all boreholes. Almost certainly most of these had developed to exploit the natural channels within the dilated rock mass following the initial failure in the 1940s, initiated by quarrying below the natural slope. Locations of pipes logged in some of the boreholes are shown in Figure 70. Examples of the pipe sediments are shown in Figure 71.

As well as the extensive natural pipe system, evidence of previous movement was seen in exposures. Sketches of old displaced features are given in Figures 72 and 73 (sketches by Dr Paul Jennings). It is of interest that the 1997 failure did not follow the previous failure surfaces (probably reflecting that the residual strength of displaced grade V granite is not much lower than that of intact material).

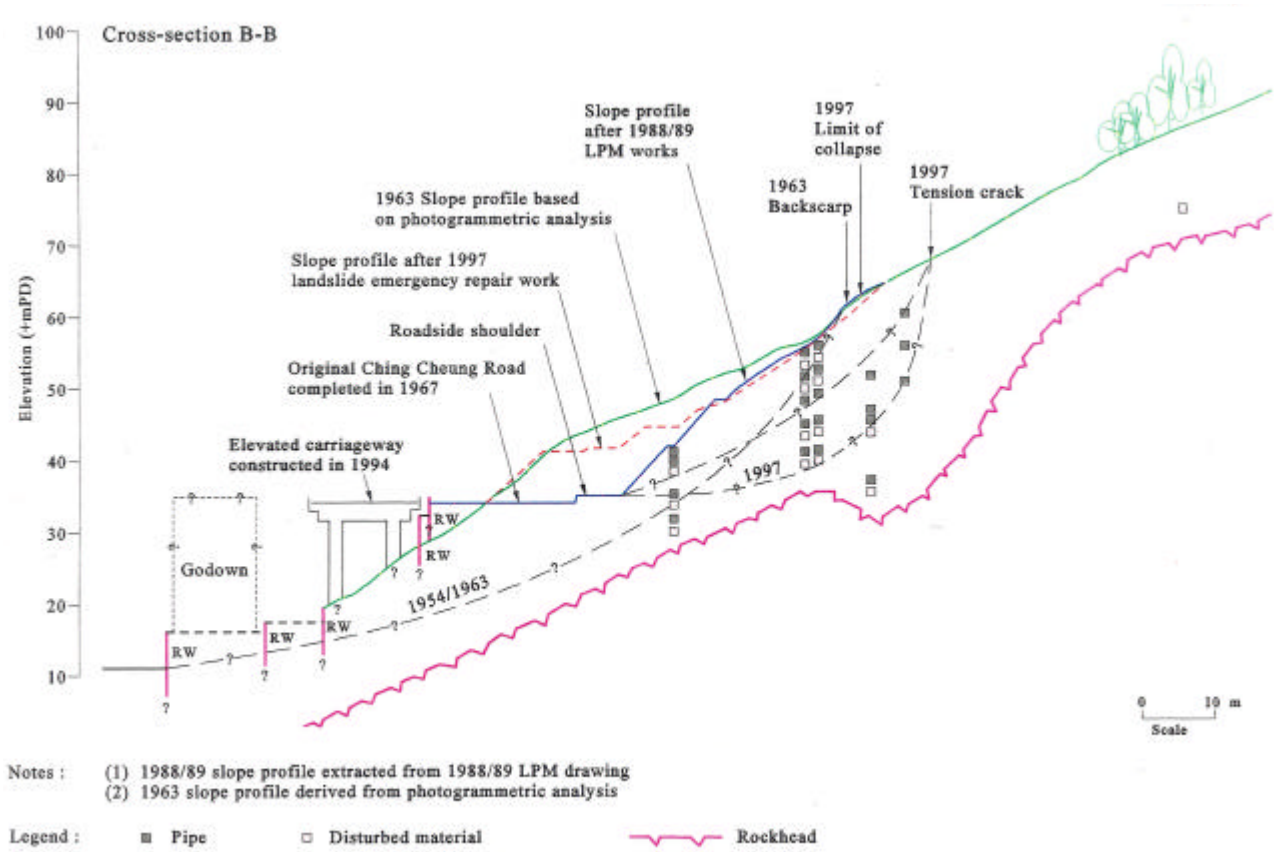


Figure 70 Ground investigation indicating the locations of natural pipes infilled with alluvial sediments

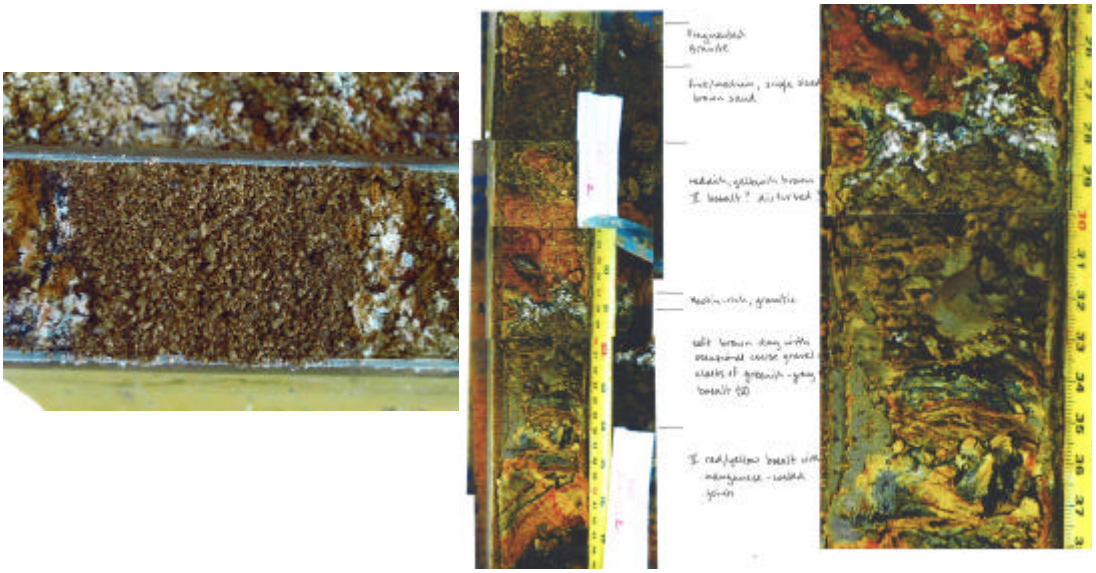


Figure 71 Left side above, single-sized sand with decomposed granite above and below. Right side, gap-graded clay with gravel, overlying decomposed basalt dyke.

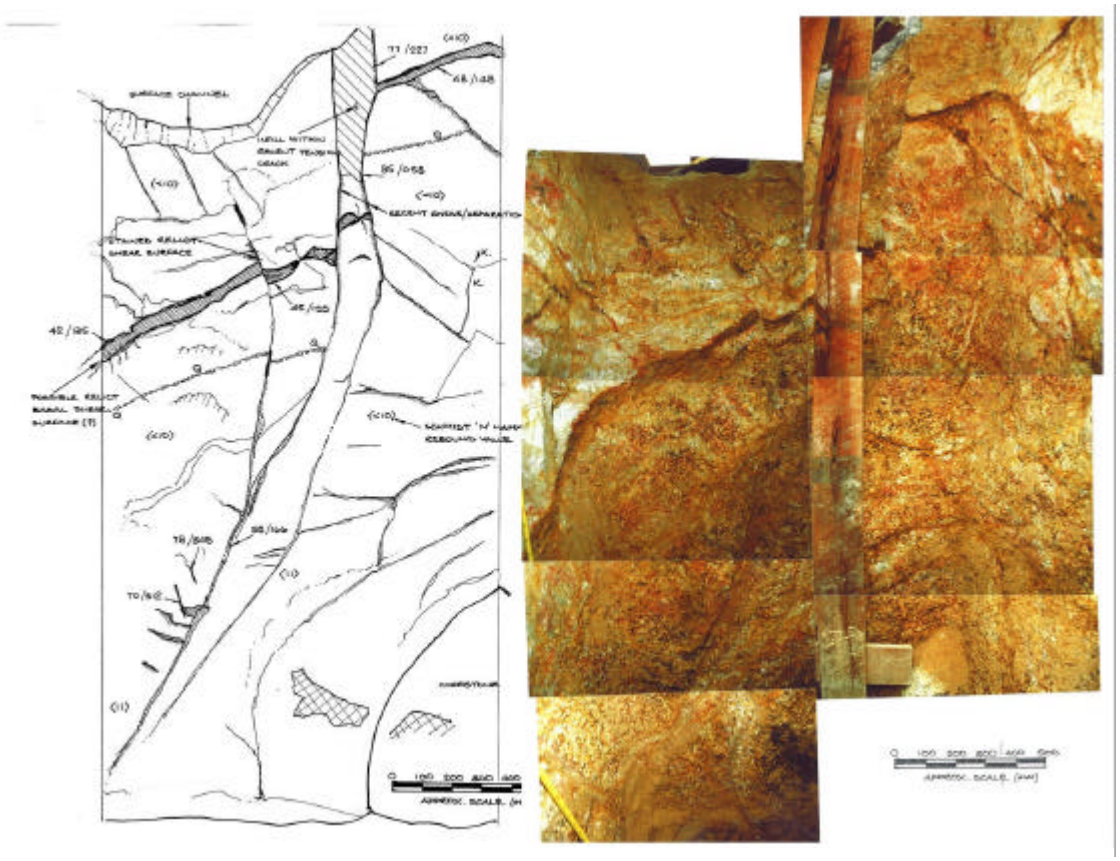


Figure 72 Old, displaced discontinuities within the failed mass at the Ching Cheung Road landslide. Note lack of reactivation of these displaced features during the 1997 failure.

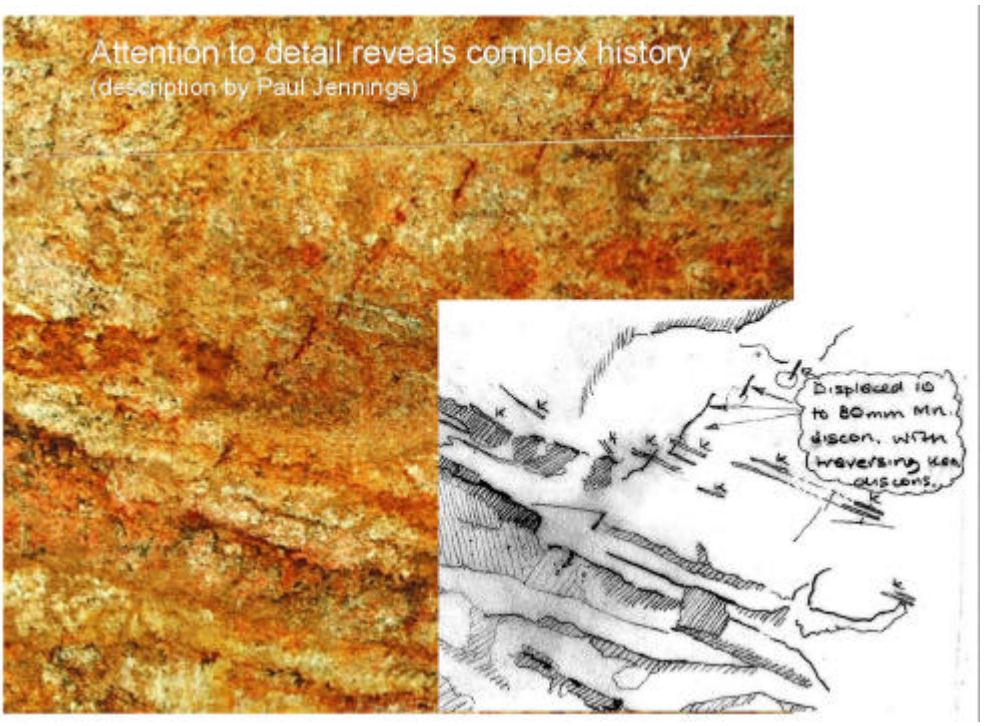


Figure 73 Further evidence of previous movements within the failed rock mass

CONCLUSIONS

In this paper it has been shown that weathered profiles are not static but develop and evolve over time.

Some major aspects such as general profile development and fracture proliferation probably occur over thousands or millions of years. Other entropic processes that are generally detrimental to engineering performance take place over much shorter time spans. These processes include internal erosion of material and progressive failure.

Weathering is primarily a process of weakening and the way that it takes place leads to heterogeneity. The weakening process is still very poorly documented so that engineers routinely use poorly representative parameters in their models. Description of weathered rock, both at the material and mass scales, is not straightforward. The current oversimplified approach and lack of fundamental research means that it will be a long time before weathered rock problems are dealt with in a proper scientific manner in the way that is now normal good practice for many other types of soil and rock.

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